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THE CONSTRUCTION OF AN EMBANKMENT WITH FROZEN SOIL.(U)
MAY 80 J J BOTZ, W M HAAS
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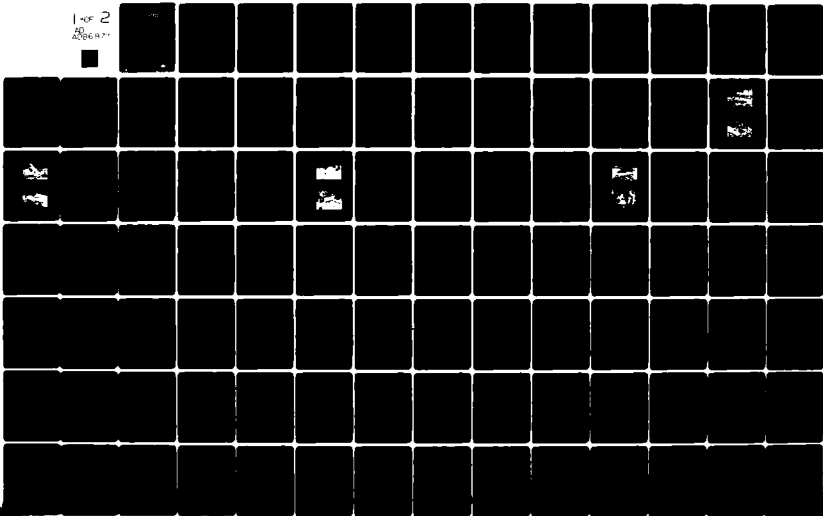

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THE CONSTRUCTION OF AN EMBANKMENT WITH FROZEN SOIL

J.J. Botz and W.M. Haas

UNITED STATES ARMY
CORPS OF ENGINEERS
COLD REGIONS RESEARCH AND ENGINEERING LABORATORY
HANOVER, NEW HAMPSHIRE, U.S.A.

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This paper presents the construction procedure, data and analysis from an experimental field program to determine the rippability and compaction characteristics of frozen soil. Also investigated was the stability upon thawing of the frozen soil compacted in the field.

A test embankment, 60 feet by 20 feet and 4.5 feet high, was constructed of two sandy soils by conventional construction techniques. The material was ripped with a tractor-mounted ripper, hauled with a dump truck and compacted with a sheepsfoot roller. The embankment was instrumented for settlement and soil

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temperature.

Field and laboratory testing was conducted to analyze the deformation in the embankment with an elastic method and a finite element computer solution. Also the frozen chunks, produced in the ripping operation, were analyzed to determine the rippability of the frozen ground.

From the results of the experimental program, several important conclusions concerning winter earthwork were obtained. (1) Ripping frozen soil can be accomplished with heavy equipment which will produce a large range of chunk sizes. (2) The effectiveness of field compaction of frozen material is highly dependent on the moisture content of the soil, AND (3) The magnitude of settlement in embankments constructed of frozen material is closely related to the compacted dry density of the placed soil.

PREFACE

This report was prepared by James J. Botz and Professor Wilbur M. Haas, Dept. of Civil Engineering, Michigan Technological University.

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NOTATION

a_v = Coefficient of compressibility, ft.^{2}/lb.

E = Young's modulus of elasticity, tsf.

E_c = Constrained modulus of elasticity, tsf.

e_o = Initial void ratio.

Δe = Change in void ratio.

G_s = Specific gravity of the solids.

H = Maximum height of embankment for a given base width and side slope, feet.

H' = Height of sample, feet.

$\Delta H'$ = Change in height of sample resulting from thaw, feet.

h = Distance above datum to the top of embankment, feet.

I = An influence factor, a function of the embankment dimensions.

W = Moisture content, %.

z = Distance above datum (base of embankment) to the point of interest, feet.

γ = Unit weight of embankment material, tcf.

γ_d = Dry unit weight, pcf.

γ_{d_1} = Thawed dry unit weight, pcf.

γ_{d_2} = Frozen dry unit weight, pcf.

$\Delta\varepsilon_v$ = Change in vertical strain, feet.

μ = Poisson's ratio.

ρ = Single lift displacement at centerline of the embankment.

$\Delta\sigma$ = Change in stress, psf.

σ_v = Vertical stress, tsf.

I. INTRODUCTION

There is a continuing interest in the development of construction methods for winter conditions. These interests stem from the many benefits which are associated with winter construction. Desirable attributes of winter construction include earlier project completion, reduction of seasonal unemployment in the construction industry and the development of natural resources regardless of the season.

Earthwork is usually a critical part of winter construction. The excavation problem has been addressed with some reasonable success and it is practical to excavate and move frozen soil in many situations (25)*. It is in the placement of the soil, however, that winter construction is most restricted. All state and federal agencies at present either prohibit frozen soil placement or limit such action so as to make it uneconomical to do so. Yoakum (42) has researched the policies of agencies concerning cold weather earthwork and reported that:

> "Twenty-five of the forty-five highway departments which replied to the questionnaire stated they do not construct embankments using frozen soil during freezing weather and they do not allow footings or pavements to be placed on frozen ground."

These restrictions and limitations developed from the observation that frozen soil is difficult, if not impossible, to compact to a density that would be rather easily obtained during the summer. Therefore, the fear exists that the low densities will result in excessive settlements and/or loss of stability.

Realizing the need for field experimentation in the area of cold

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*Note: Numbers in parentheses refer to publications listed in Appendix A, References.

weather earthwork, the U.S. Army Cold Regions Research and Engineering Laboratory sponsored the construction of an experimental embankment to be constructed of frozen material using conventional construction techniques. An embankment 60 feet by 20 feet and 4.5 feet high was constructed during February and March of 1975. It was built of frozen material which would have been favorable as an embankment material under normal conditions. Adequate instrumentation to determine the temperature and displacement characteristics of the embankment was installed. During and after construction supplemental laboratory tests were conducted to obtain sufficient information to characterize the soil and to develop its unfrozen compaction and compressibility characteristics.

The primary objectives of the field work were to determine, first-hand, the practical problems involved in winter earthwork and to obtain sufficient data to assess the effectiveness of field compaction and ripping during cold weather. In addition, deformations of the embankment were to be monitored and related to the field compaction test results.

II. LITERATURE REVIEW

Earthwork is a critical part of any construction activity and includes excavation and placement of soils. Present techniques for cold weather activities of both phases of winter earthwork are discussed below.

2.1. Cold Weather Excavation

Excavation of frozen soils has been conducted with reasonable success and Yoakum (41) found that large scale excavations of frozen soil deeper than 2 feet were completed by either ripping with tractor-mounted rippers or blasting with explosives. Tractor-mounted rippers have proven to be most efficient. Cross ripping is generally used, with the spacing and number of passes dependent on the soil conditions and the desired chunk size.

Ripping of frozen soil is fairly common and Haley (12) reported on ripping operations which proved to be efficient for frozen soil excavation. Luhr Brothers Construction from Columbia, Ill. ripped frozen clay with a D9 bulldozer in temperatures that dropped to -6°F. Contractors Lee and Fox from Lexington, S.C. broke up 250,000 cubic yards of frozen material on a highway project in 1958. In addition, there has been extensive testing of rippers on the Mesabi Iron Range in northern Minnesota conducted by Caterpillar Tractor Company. Caterpillar D9 bulldozers equipped with 8 foot Kelly rippers have been used to effectively rip overburden material frozen to a depth of 6 feet. The toughest material to rip in the frozen condition was a sandy clay. It was estimated the material was excavated at half the drilling

and blasting cost. One borrow pit was excavated for 3 cents per cubic yard less than the cost of the powder alone.

2.2 Placement of Frozen Ground

2.2.1 Specifications

Placement of soil in winter is difficult because the low densities obtained from compacted frozen soils will result in excessive settlement or loss of stability. In fact, most government agencies state that frozen soil shall not be used in the construction of embankments, and embankments shall not be constructed on frozen ground (42). Some state agencies are more liberal and allow frozen soil placement to a limited degree. Maine permits embankments to be composed of frozen ground when the depth of fill and the depth of frozen ground is less than 5 feet. In addition, Maine allows placing material on frozen ground if the subgrade was compacted prior to freezing (42). Wisconsin allows embankments constructed of granular material to be formed in the late fall and early winter (42). Some states will permit the placement of frozen ground, but their specifications and requirements make it uneconomical and impractical to do so. The Canadian Good Roads Association states "normally contractors are not allowed to place frozen ground materials in highway fills" (42). Exceptions to this are when large fills are to be constructed of clean granular material of low moisture content and when fills will be left to consolidate for a year or more.

2.2.2 Compaction of Frozen Soils

Compaction of frozen soil is difficult because the voids in the frozen soil are occupied by ice. Because of the cementing effect of ice, frozen soils exhibit a high amount of cohesion and thus resist compaction. The magnitude of the cohesion depends on the composition of the soil, its

moisture content, and the structure and temperature of the ice (25). Research results indicate granular soils can probably be adequately compacted even when frozen (25,14). However, research has verified that when soil temperatures reached 20°F to 25°F it was extremely uneconomical and impractical if not impossible to achieve specified densities(25).

The compaction characteristics of a silty sand at -7°F were investigated by Haas, Alkire and Kaderabek (11) and their results are presented in Figure II-1. The dry density decreases rapidly at very low water contents and at water contents greater than 3% it decreases at a slower rate. At zero water content the density was the same as that of soil compacted at above freezing temperatures. Temperature conditions have a great influence on the compaction results if the natural moisture content of the soil exceeds 3% (5). Thus, a sandy gravel could be placed effectively in the frozen condition if the moisture content is very low.

Sallburg and Johnson (16) showed similar results for the effect of freezing temperatures on the compaction of a gravelly sand. They showed a drop in unit weight of 120 pcf to 98 pcf when the compacting temperature was reduced from 74°F to 20°F at a moisture content of 10% with Standard AASHO Compactive Effort.

2.2.3 Compaction Methods

Different methods of compacting frozen soils were investigated in connection with the construction of the Messaure Dam in northern Sweden (5). The Swedish State Power Board found that heavy vibrating equipment was most suitable for compacting frozen granular soils. A 4.1 ton vibratory sheepsfoot roller and a 3 ton vibrating roller were tested for their compactive efforts. For gravel having a moisture content of 2 to 3%, the degree of compaction after 4 passes was 80 to 85%

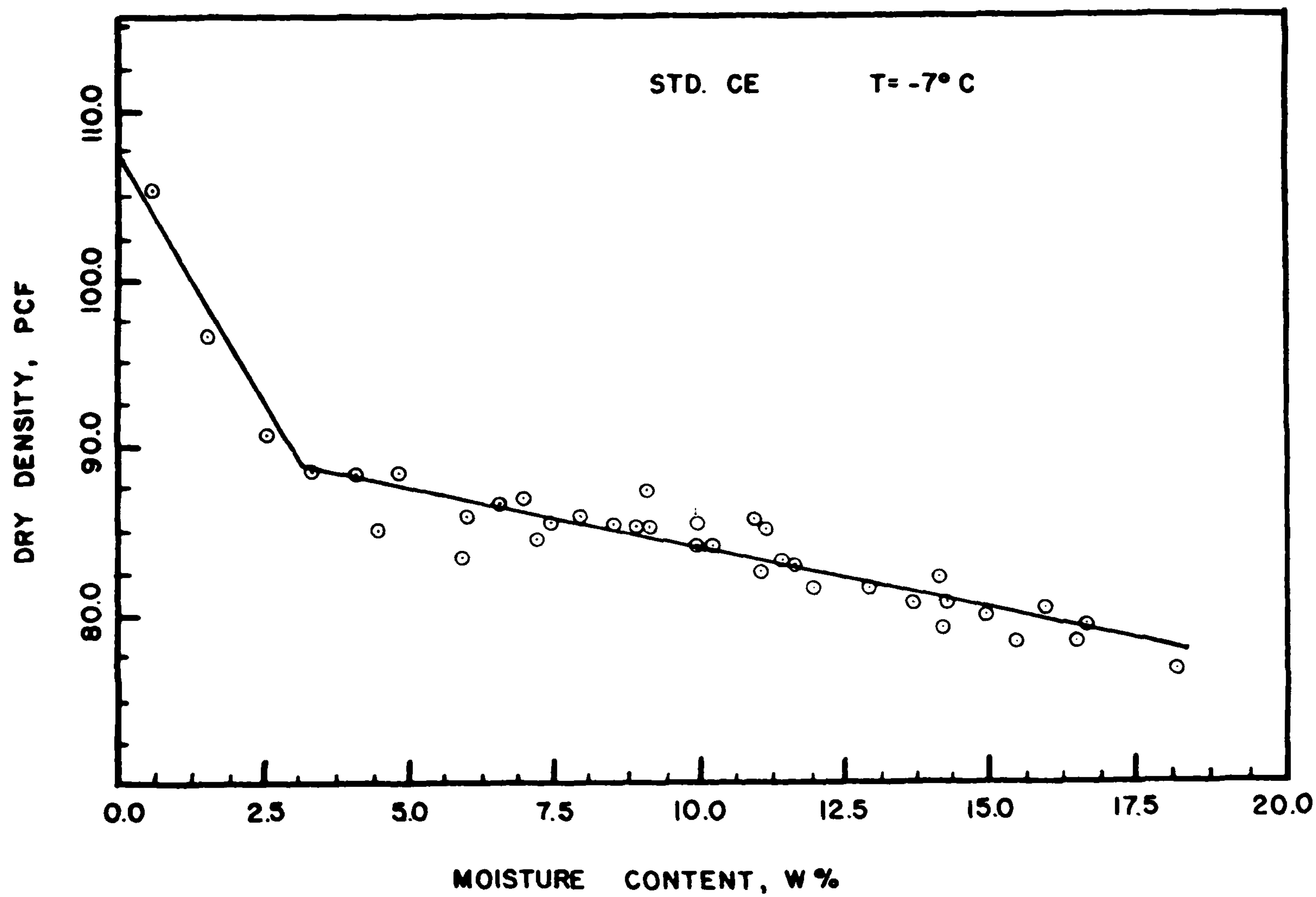

FIGURE II-1 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT (HAAS, ALKIRE AND KADERABEK).

of the Modified Proctor value determined on unfrozen soil. They found that the water content and the duration of the compacting method were the two chief factors which determined the effectiveness of the equipment on a given soil (5).

Bernell (4) presented specifications for placing and compacting winter fills as follows:

1. Snow and frozen soil should be removed from the site to allow the fill to be placed on unfrozen ground.
2. A compacted gravel filter should be used between the fill and the subgrade material to insure adequate drainage.
3. Placement of embankments should not be permitted during snow fall.
4. Each lift should be clear of snow and ice before a new lift is placed.
5. Final lifts should not be placed until the fill is completely thawed to prevent permafrost in large embankments.

2.2.4 Compaction Equipment

The effectiveness of all compaction equipment is dependent on the characteristics of the equipment and the type of soil being compacted. Cohesive soils generally compact best with kneading-action type rollers such as sheepsfoot rollers. With less cohesive soils, the sheepsfoot roller becomes less effective, and with a completely cohesionless sand the sheepsfoot roller will tend to disturb rather than compact the material (15). Frozen soils, when ripped, produce aggregates or chunks of individual soil particles that exhibit little cohesion. Therefore, it would appear that methods used with cohesionless soils would be most efficient in compacting frozen material. However, this observation is not supported by any field data and kneading types of compaction may prove

to be effective with frozen soils.

Apart from the type of soil being compacted, the effectiveness of a sheepsfoot roller is primarily dependent on the contact pressure, which is a function of the weight of the roller, area and shape of feet, and the number of feet, (the contact pressure of a sheepsfoot roller is determined by dividing the weight of the roller by the area of one row of tamping feet). Tamping feet of 7 sq. in. are generally recommended; however, feet 50% larger are more effective in silty or sandy soil (1). The length of tamping feet has little effect on the compactive effort of the roller but is critical for the stability of the roller.

The contact pressure should be as large as possible but is limited by the bearing capacity of the soil. If the bearing capacity is exceeded, the roller will: a) sink, reducing the contact pressure and the efficiency of compaction, or b) produce a constant shearing of the soil and little compaction (1).

Alternate methods of determining the effectiveness of a sheepsfoot roller are by the percent of coverage or compactive effort. The percent coverage is equal to the total foot area divided by the area of an imaginary drum with a diameter equal to the distance between diametrically opposite feet. The compactive effort (ft.-lbs.),on the other hand, is approximately the drawbar pull (lbs.) times the number of roller trips over each vertical foot of compacted fill divided by the roller width (ft.). The drawbar pull can be assumed to be 25% of the roller weight for a sandy soil and 40% of the roller weight in clayey soils (33).

Lift thickness and number of passes also influence the effectiveness of any given piece of compaction equipment. Generally, the heavier the roller the greater the lift thickness that can be placed, with 9 to 12 inches recommended for sheepsfoot rollers.

Allen (1) has found that the relation between compacted density and the number of passes of a sheepsfoot roller is approximately a straight line when density is plotted on an arithmetic scale and the number of passes plotted on a logarithmic scale. Adequate compaction is usually accomplished with a maximum of 6 to 10 passes of the compaction equipment. An increasing number of passes beyond this amount usually proves uneconomical (20).

2.3 Settlement of Thawing Soils

When frozen soils thaw, water is released and settlement develops as the water is squeezed from the pore spaces and stress is transferred to the soil skeleton. The evaluation of the magnitude of thaw settlement has generated considerable interest and has been investigated by several researchers (8,29,38). The usual method of evaluation of thaw settlement is to assume one-dimensional thawing of a uniform, homogeneous frozen soil and then predict settlement from equations developed by modifying Terzaghi's Theory of Consolidation.

A more fundamental approach to thaw-settlement has been developed by Crory (8). He models frozen soil in terms of volumetric relationships between the soil, ice, water and air present in a frozen soil and notes that a thawing soil can either expand, experience no change in volume, or consolidate.

From these sample boundary conditions a general solution to determine the change in volume of a frozen sample upon thawing is developed. The solution involves only the frozen and thawed dry unit weight of the sample and can be expressed as:

$$\frac{\Delta H'}{H'} = \frac{\gamma d_2 - \gamma d_1}{\gamma d_2}$$

where γ_{d_1} is the frozen dry unit weight, γ_{d_2} is the thawed dry unit weight, H' is the height of the sample and $\Delta H'$ is the change in height of the sample resulting from thawing. A positive result signifies settlement of the sample and a negative result signifies expansion of the sample.

The expression developed by Crory is based on one-dimensional consolidation and for more generalized states of stress may significantly over-estimate thaw-settlement. For three-dimensional consolidation Luscher and Afifi (26) have shown that axial strain developed during thaw under isotropic stresses is approximately 1/3 of the value for one-dimensional thaw, but anisotropic (K_0) thaw consolidation resulted in strains that were equal to one-dimensional strains.

2.4 Settlement of Embankments - Elastic Methods

Embankment settlements cannot be accurately predicted using a one-dimensional analysis because lateral translation occurs during and after construction. However, since an elastic analysis incorporates not only deformation in the direction of the applied normal stress but also considers the lateral deformation resulting from the normal stress, several researchers have investigated the elastic settlements of embankments. Poulos and Davis (31) have summarized their results in the recent book, Elastic Solutions for Soil and Rock Mechanics.

Clough and Woodward (7) have shown that the calculated displacements in an embankment are strongly affected by the compaction process and methods of analysis. The incremental construction procedure of evaluating stresses and displacements will result in the maximum vertical displacements occurring at midheight where the region is affected by all the strains developed below this level due to all the load applied above

this level. At the end of construction an incremental analysis will show zero vertical displacement at the top, for after the top material is placed no further strains are developed to result in any deflections.

The single-lift analysis assumes that the stress and deformation can be obtained by direct application of the gravitational forces on the complete structure. "Most embankments are constructed by an incremental process and the loading is accumulated gradually during construction" (7). The single-lift method does not take into account the construction method and is valid only if the state of stress is statically determinate at all stages of the construction process. The single-lift method will, however, give a satisfactory approximation in most cases (7).

III. FIELD EXPERIMENT CONSTRUCTION SEQUENCE

In late January 1975 a field project was initiated to study ripping and placing frozen material. A basic construction plan was developed that would utilize personnel of Michigan Technological University and equipment from the Keweenaw Research Station.

3.1 Site Selection

The project was initially conceived as a field verification of a laboratory study on the compaction of frozen soil (11). The test soil used in the original test program was a silty sand obtained from a borrow area at the Houghton County Airport. An attempt was made to place the field test near this borrow area so that the same material could be used. However, transportation of equipment to the site became a major factor in the site selection and sites closer to the Research Station were investigated.

The site selected was within 1000 feet of the Research Station and had several other distinct advantages: 1) It was directly on the centerline of a proposed roadway and any construction activity could be integrated into future road construction plans. 2) Suitable material was located in a borrow area 400 feet from the site. 3) The site was previously used as a borrow area and any construction activity would have little impact on the existing environment. 4) The Houghton County Airport Authority was willing to permit work in this area with a minimum of restrictions. A general plan of site is shown in Figure III-1.

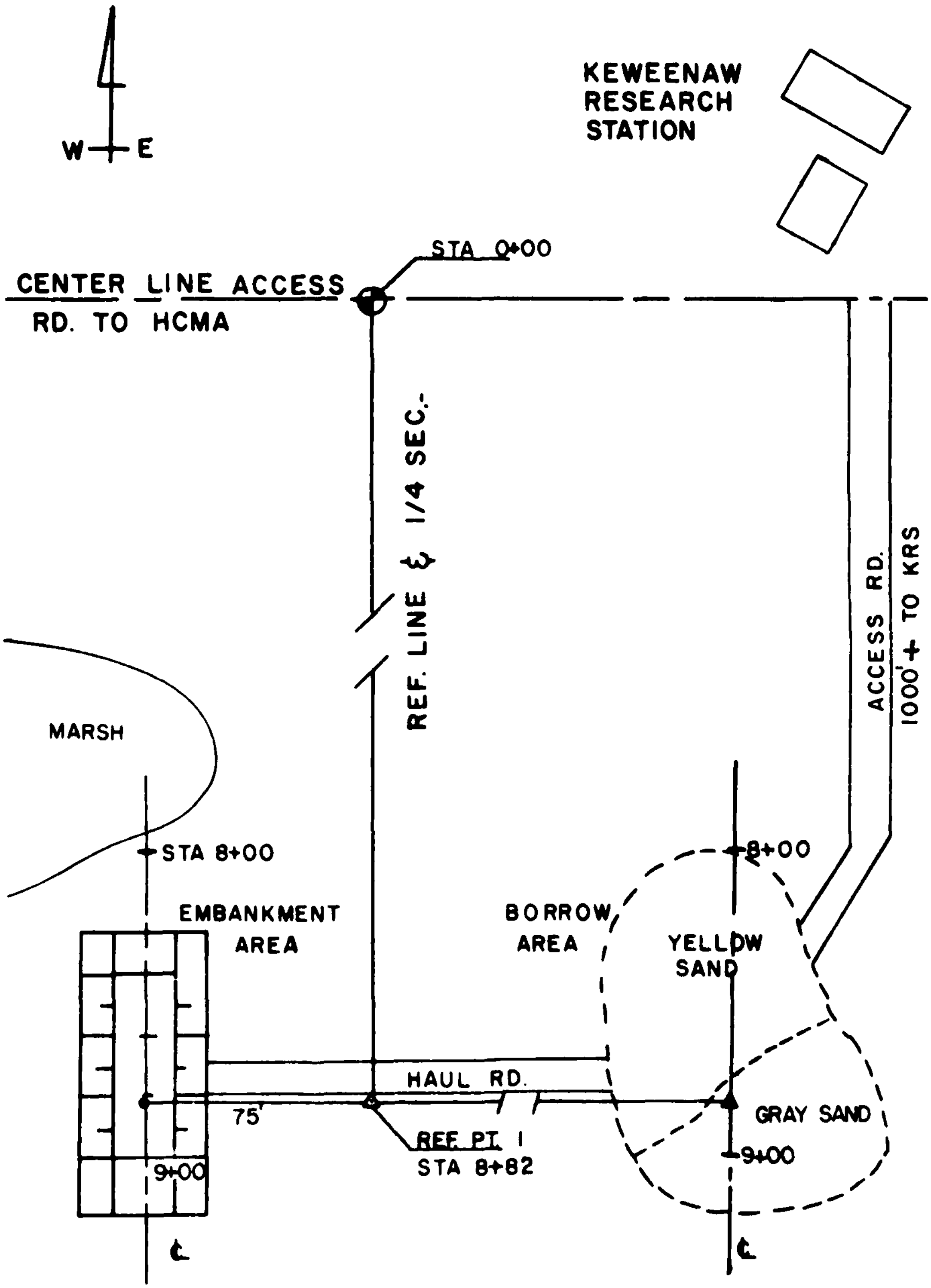

FIGURE III-1 GENERAL PLAN OF SITE.

3.2 Layout and Preliminary Work

The preliminary field work was started in mid-February with tentative locations selected for the embankment and borrow areas. A project reference line was established 75 ft. east of the proposed centerline of the road. This reference line was used to establish the centerline of the fill. Two other reference lines were run from the project reference line to establish the centerline in the borrow area. All of the preliminary soil borings were referenced to these lines.

During preliminary work the snow varied in depth from 1 to 4 ft. which insulated the ground sufficiently to keep the soil unfrozen. Therefore, hand auger borings were made with little difficulty. The borings in the borrow area revealed 2 types of soil similar in gradation but different in color. It was decided that both soil types would be used for the ripping and embankment construction with the soil types kept separate within the fill.

Existing in the fill area was a swampy region to the north of Station 8+00 with the terrain sloping towards this general direction. Borings in the fill area revealed a uniform poorly graded sand from Station 8+40 to 9+40. A highly organic silty clay soil was found from Station 8+40 to 8+00 and was judged to be unsuitable for the subgrade of the fill. Based on the soil borings and the wet area to the north, the fill was to be placed south of Station 8+40 and was to extend approximately 100 ft.

After the site selection was completed, the outer boundaries of the fill and borrow areas were flagged to aid in the snow removal.

3.3 Snow Removal

As already stated, the accumulation of snow in the area adequately insulated the ground to prevent frost penetration into the soil. However,

some areas were wind swept and clear of snow which resulted in ground frost in localized areas.

Therefore, to insure an adequate amount of frozen soil for ripping, the borrow area was cleared of all snow. A D7 bulldozer was used in this process which was performed twice during the month of February to keep the area free from snow.

3.4 Ripping

The ripping operation of the borrow material started on March 3, 1975. A single tooth ripper mounted on the hydraulically operated blade of the D7 was used for the ripping (Figure III-2A). Attempts to rip the soil prior to this date resulted in failure as the ripper tooth could not penetrate the frozen crust. However, on March 3 the temperature had risen slightly and the D7 proved to be adequate for ripping the soil.

No set number of passes or spacing of passes was used in the ripping operation. The tractor made passes in any direction where it could penetrate and rip the soil. Eventually all the material with a frost penetration of less than 1 foot was ripped. Most of the soil which could not be ripped was near Station 8+80 where the snow cover had been minimal. The yellow sand was much easier to rip than the dark sandy soil which can be partially accounted for by the higher moisture content and density of the gray sand.

The chunks produced from the ripping were irregular in shape with the large chunks being platy and limited in least dimension by the depth of frost. Frozen particles too large to be loaded with the front end loader were pushed to the side and not used in the fill. The rest of the material was stockpiled in such a manner that the yellow and gray sands could be moved separately to the fill area.

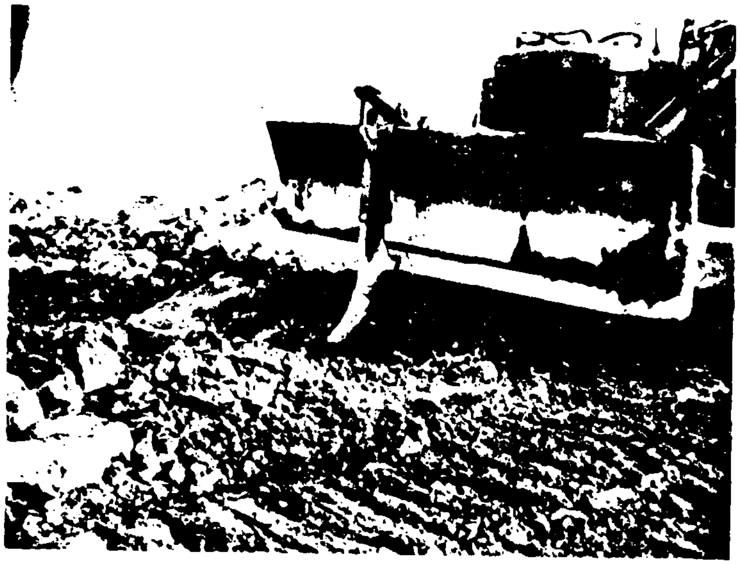

FIGURE III-2A D7 EQUIPPED WITH RIPPER TOOTH

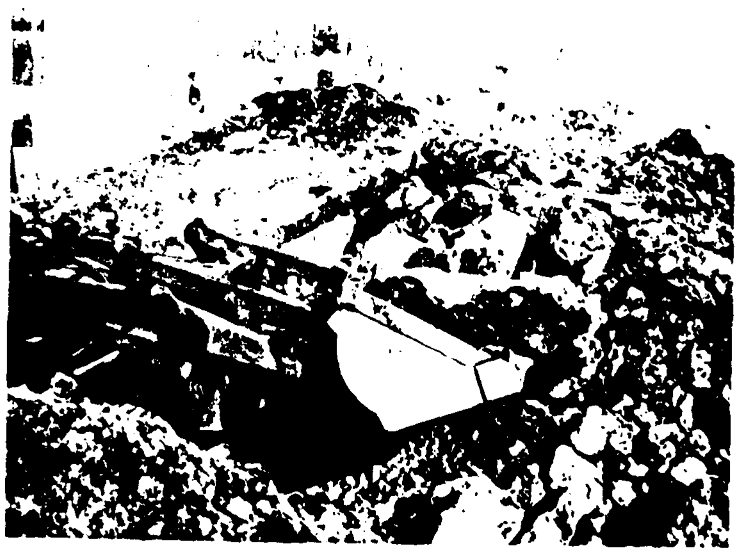

FIGURE III-2B LOADING OF FILL MATERIAL

3.5 Hauling and Placing

Once an adequate amount of borrow material was stockpiled for the first lift of the embankment, the hauling operation started. The material was loaded into a 4 cubic yard dump truck with an Allis-Chalmers HD-50 TraxCavator (front end loader) and dumped at the desired location in the fill area (Figures III-2B and III-3A). The yellow sand was placed first from Station 8+40 to Station 8+80 and the gray sand was placed from Station 8+80 to Station 9+00. When enough material was hauled for one lift, it was leveled with the D7. Approximately 18 inches of material was placed in each lift to produce a 12 inch compacted lift. Control of the lifts and fill elevations was maintained with level shots taken to various points on the embankment.

3.6 Compaction

The compaction of all lifts followed the same sequence of operation. First, precompaction densities of the fill material were determined at 2 locations. The lift was then subjected to four passes of the sheepsfoot roller followed by another set of density determinations. Each pass of the roller covered the entire fill area, but as the side slope converged to the center only the area within the slope lines of each layer was compacted.

The primary piece of equipment used to compact the frozen soil was a sheepsfoot roller rented from a local contractor (Figure III-3B). The roller was 4 feet wide and 3 feet 6 inches in diameter. It had a total of 64, 3 inch by 4 inch, club type feet which gave a 9.1% coverage. The roller weighed 3300 lbs. The total weight of the roller was increased to 7368 lbs. by filling the drum with a solution of water and $CaCl_2$ and by adding thirty 80 lb. concrete blocks to a specially constructed

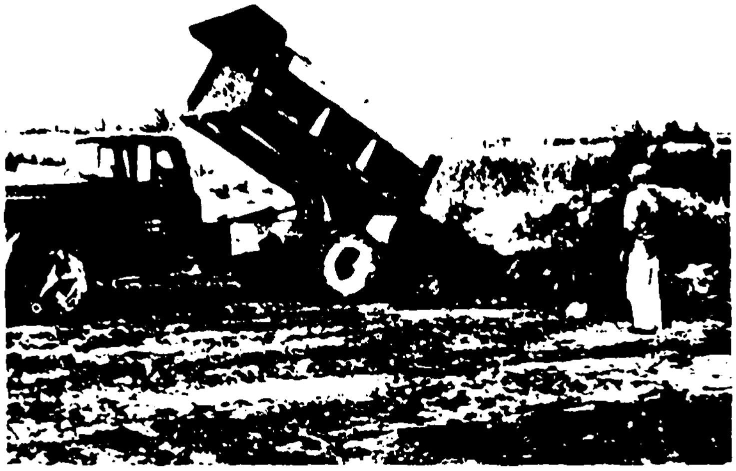

FIGURE III-3A PLACING OF FILL MATERIAL

FIGURE III-3B COMPACTION EQUIPMENT

outrigger frame. The total weight resulted in a maximum contact pressure of 154 psi.

In most cases, the sheepsfoot roller was pulled by the D7, but occasionally it was more efficient to have the D7 ripping material for the next lift at the same time compaction was being conducted. For these cases the front end loader (TraxCavator) was used to pull the sheepsfoot roller.

3.7 Instrumentation

One of the objectives of the project was to correlate rate of settlement with the rate of thaw in the embankment. This required some method of obtaining settlements at various locations within the fill. Thus, settlement plates were installed at existing ground level, and at 1/3 and 2/3 of the final fill height (Figure III-4). The plates were 12 by 18 by 3 inch concrete slabs with a bolt embedded in the center of each plate for use in obtaining the elevation of the plate. As the fill was constructed the plates were placed at the desired location in hand excavated pits and placed on 1/2 inch of unfrozen sand. Initial elevations were taken followed by backfilling and placement of the next lift. When the final lift had been placed, bore holes were driven with hand augers to the plates and 1½ inch PVC pipes were installed as access tubes. Due to the difficulty in boring through the frozen material, several of the lower plates were not located. Also, due to a slight skew in the alignment of the fill, not all the settlement plates had the planned amount of fill cover.

Surface markers were placed at five stations perpendicular to the centerline of the fill (Figure III-4). The purpose of these markers was to provide a consistent location point for obtaining cross sections. The

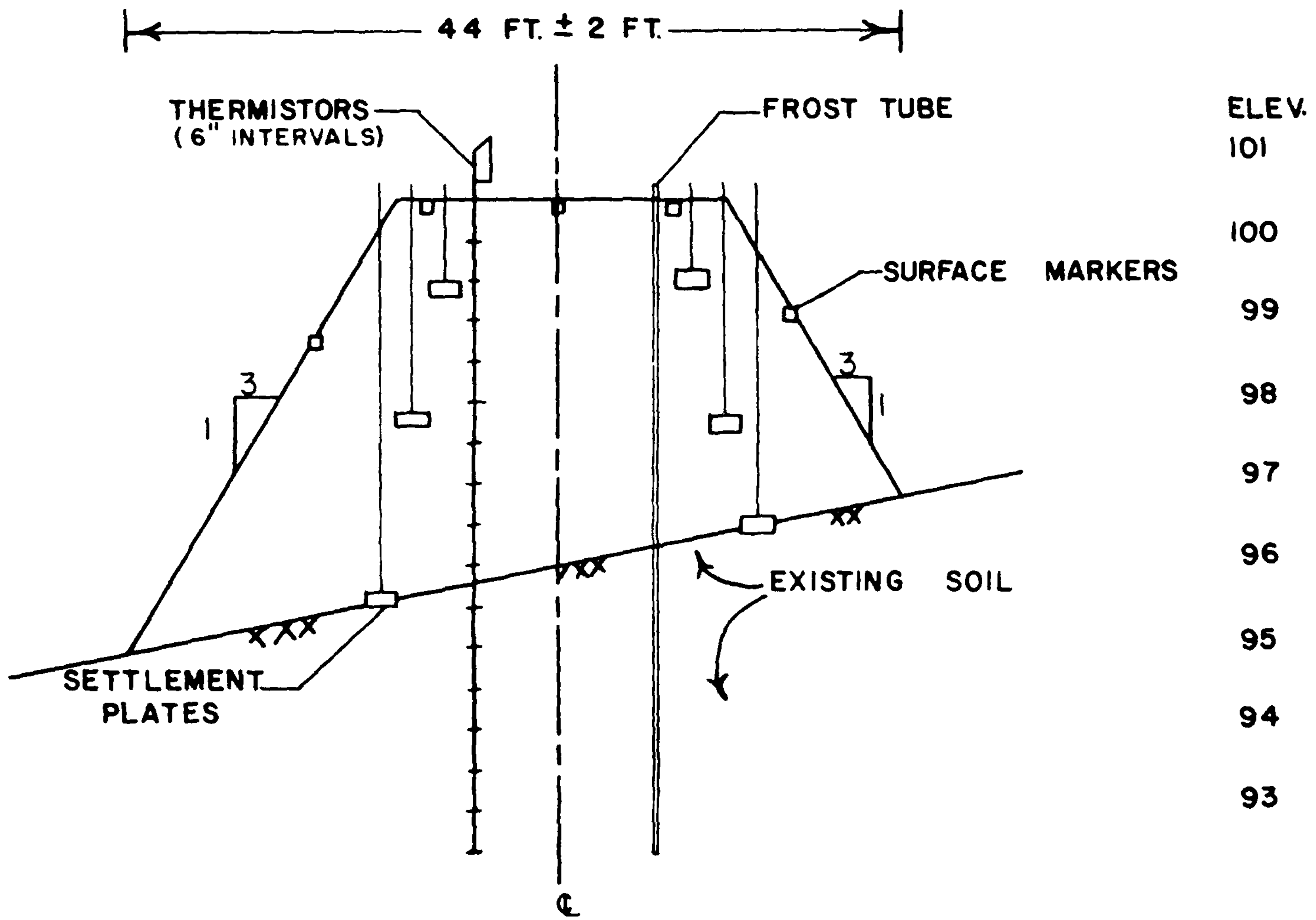

FIGURE III-4 TYPICAL CROSS-SECTION OF INSTRUMENTATION.

markers were concrete cubes, 4 by 4 by 4 inches, with a nail embedded in the center. Elevations at the top of the nails were taken at regular intervals.

Six frost tubes and two sets of thermistors were installed to monitor the temperature of the embankment (Figure III-4). Each tube consisted of an 8 ft. long flexible Polyurethane tube of 1/2 inch I.D. The tube was attached to a piece of 3/4 inch cove molding. The tube was filled with a methyl blue solution which upon freezing would turn from blue to a clear-white. Therefore, the frost penetration could be measured by the length of clear-white frozen solution. Measurements were read from scales attached directly to the polyurethane tubing.

Installation of the frost tubes required bore holes through the fill and into the subgrade. A track mounted drill rig was rented for this purpose, after a hand held power auger proved to be inadequate. A 6 inch diameter bore hole was drilled and a 1½ inch I.D. PVC pipe was installed for an access pipe. The soil was backfilled and compacted around the PVC pipe and the frost tubes placed inside the pipe.

Because the accuracy of the frost tubes was uncertain and considering they had not been field tested, two sets of thermistors were installed to insure accurate temperature data. Each set was 8 ft. long with 16 thermistors spaced at 6 inch intervals. Bore holes were drilled in the same manner as that of the frost tubes. The thermistors were taped to 1/2 inch cove molding and placed directly into the bore hole and backfilled. The thermistor leads were collected in a junction box and numbered according to their respective location in the fill (Figure III-4). Each set was placed within 3 ft. of a frost tube so that they could be checked against each other.

3.8 Construction Problems

Winter earthwork is usually considered to be hard on both equipment and personnel. During the course of the field construction temperatures were generally in the twenties, with lows in the teens on some mornings and in the low thirties during the late afternoon. Several snowfalls of 1 to 3 inches were experienced during the construction period, but they did not seriously hamper field operations. Work was not done during heavy snowfalls. However, it continued during a few brief periods when very light and fluffy snow was falling. This was considered to be acceptable because the amount of snow being incorporated into the fill was negligible. In the case of an overnight snowfall, however, the accumulation was removed from the surface of the partially completed embankment before additional soil was placed on it. Considering the environmental factors there were very few problems with the equipment. Several problems were encountered, however, that may be significant in overall evaluations of the project.

1) The D7 performed very well during its approximately 100 hours of operation. On one morning ($T = 10^{o}F$) the tractor would not start and the plugs had to be removed and heated before it would start. This resulted in a loss of 4 hours operating time.

2) The hydraulic cell for the blade had a chronic leak that progressively became worse as the project continued. This was probably aggravated, if not caused, by the ripping operation.

3) Once the dump truck became stuck when backing into the loading area. A hole was punched in the radiator of the truck by the D7 as it was being pushed out. In general, the truck was only marginal for operation in the snow and mud around the fill area.

4) The sheepsfoot roller was quite unstable on the embankment slopes

FIGURE III-5A INSTABILITY OF SHEEPSFOOT ROLLER

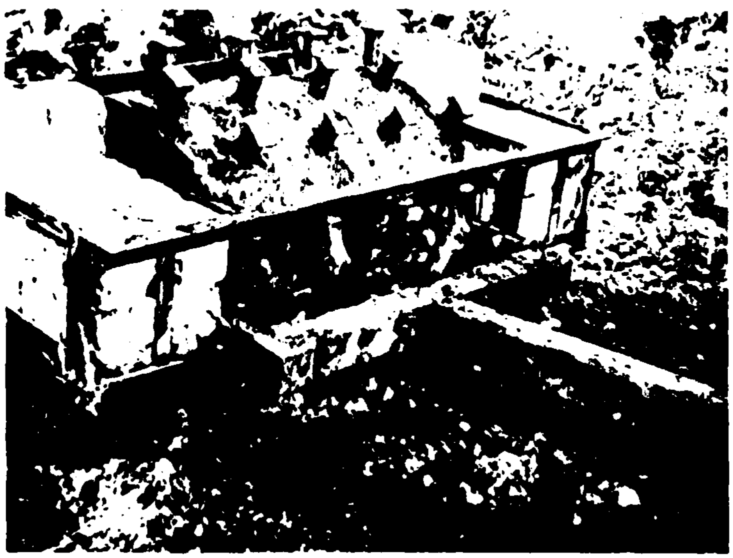

FIGURE III-5B SOIL CHUNK LODGED IN ROLLER FEET

and while passing over large chunks (Figure III-5A). This instability was due to the weight added to the sides of the roller and the relative narrow width of the roller.

5) It was anticipated that the chunks would be easily deaggregated by the action of the roller feet. However certain sized chunks became lodged between the feet (Figure III-5B). When the chunk rotated to the rakes of the roller the chunk would not break down and the rotation of the drum would be stopped. The roller would then have to be backed up and the chunk removed with a pry bar and/or pick. This caused delays in the compaction of the fill.

6) In place density tests, using the balloon method, were difficult to conduct on frozen ground. The hole had to be excavated with the aid of a chisel and hammer. The hammering did result in the loss of some material, but this was still the best method available for obtaining the as-compacted densities.

7) A soil auger (Little Beaver) was to be used to bore the holes for placing the frost tubes and thermistors. Due to the large number of rocks in the fill material this auger would not penetrate the frozen soil and an alternate method of drilling these holes had to be obtained.

IV. TESTING PROCEDURE

4.1 Laboratory Testing

Laboratory analysis of the soils obtained at the construction site was conducted at the Soil Mechanics Laboratory, Michigan Technological University. Testing of the soil samples was conducted using American Society for Testing and Materials (ASTM) methods, with some deviations as noted below.

4.1.1 Classification

The embankment soils were classified with a gradation analysis of the soils conforming to ASTM D422 and the liquid and plastic limits of the soils conforming to ASTM D423 and D424

4.1.2 Compaction

The procedure used in the laboratory to determine the moisture-density relationships of the soils conformed to ASTM D698 and D1557. In addition to the Standard and Modified Proctor compaction tests, a test was also run at a non-standard compactive effort. The procedure used conformed to ASTM D698, except the sample received 25 blows per layer instead of the Standard 56 blows per layer for a 6 inch mold. The decrease in the number of blows resulted in a compactive effort for the non-standard test of 5500 ft.-lbs. per cu. ft. The Standard Proctor test delivers a compactive effort of 12,375 ft.-lbs. per cu. ft. and the Modified Proctor test delivers 56,258 ft.-lbs. per cu. ft.

4.1.3 Frozen Compaction

A series of compaction tests on frozen samples of the embankment soils were conducted in the cold room laboratory. These tests were performed using Standard Proctor compactive effort with a 6 inch mold.

Details of sample preparation are presented by Haas, Alkire and Kaderabek (11), and summarized in Appendix I.

4.1.4 One-Dimensional Consolidation

The procedure used to determine the consolidation properties of the soils conformed to ASTM D2435. The soil was sieved through a #4 Standard Sieve and compacted in the consolidometer mold in 5 equal lifts. The soil was compacted with a flat-end 1 inch diameter wooden dowel. The lifts were tamped equally to obtain samples at dry densities in the range of 85 pcf to 110 pcf at a water content of 15%.

At low values of stress (0.015 tsf - 1.880 tsf) a Soiltest Consolidometer Model C-210 was used with dead weights providing the loads. At higher values of stress (0.156 tsf - 20.000 tsf) a Karol-Warner Consolidometer Model 351 was used with a Karol-Warner auxiliary gage Model 3501.

4.2 Field Testing

Certain tests on the soils were performed directly in the field to determine the physical properties of the material in its frozen state. Samples for the various tests were obtained from various locations throughout both the borrow and fill areas. Location of the samples and other relevant material will be discussed in the description of the tests.

4.2.1 Particle Size Analysis

The determination of the particle sizes for the frozen material presented a number of problems. It was known that ripping would produce a range of particle sizes from 4 feet in effective diameter to millimeter sized particles. Obviously no set of standard sieves could be used for this analysis. The method of analysis finally adopted consisted of the following set of operations: 1) select a site in the borrow area that

contained particles representative of the sizes produced by the ripping operations, 2) select a sample from the site and weigh and measure the particles smaller than 24, 18, 12, and 6 inches in diameter (least dimension), 3) sieve the remaining material through a 2 inch and a 3/4 inch sieve and weigh the material passing each sieve.

The sieving and measuring operation described above accounted for all material from the selected sites. There were a large number of chunks produced by the ripping operation that were larger than 2 feet in diameter (Figure IV-1A). These were not included in the particle analysis since they were, in general, not included in the embankment material.

The weighing operation was completed using a tripod with a scale suspended beneath. A bucket with the soil sample was attached to the scale and the weight recorded. The entire apparatus was moved from site to site as the samples were obtained and analyzed. The sieves used in sieving the material finer than 2 inches were field sieves (Soiltest CL-320) with a rocker base. Performing the sieve analysis in the cold weather was comparatively inefficient, but was completed with no serious difficulty. A photo of the equipment used in this operation is shown in Figure IV-1B.

4.2.2 In Situ Density Measurements

There are very few methods of obtaining in situ density when the soil being tested is frozen. The sand cone method (ASTM D1556) and rubber balloon method (ASTM D2167) which are commonly used with unfrozen material were considered. The balloon method appeared to be more attractive when working in cold weather, and this method was used to obtain the densities both in the borrow area prior to ripping operations and at the fill during the compaction operation.

FIGURE IV-1A TYPICAL LARGE SOIL CHUNK

FIGURE IV-1B SIEVING EQUIPMENT USED IN CHUNK SIZE ANALYSIS

The method used in the field conformed to the basic requirements of ASTM D2167. However, the water in the reservoir was replaced with a 50/50 mixture of ethylene glycol and water. The hole could not be excavated in the frozen soil without the aid of a hammer and chisel to loosen the soil. However, two men can complete this operation effectively with very little loss of soil. As the soil was removed from the hole it was placed in a plastic bag, sealed and marked. The moisture content and weight of the excavated soil were obtained in the laboratory.

The minimum test hole volume by ASTM D2167 specifications for a maximum size particle of 1/2 inch is 0.5 cubic feet. Due to the difficulty in penetrating the frozen material, volumes of this size were not attainable in the field testing.

4.2.3 Post Consolidation Densities

Post-thaw in situ densities were taken on July 15, 1975 at which time the settlement had stopped. The densities were obtained by the rubber balloon method (ASTM D2167). The densities were taken at Station 8+60, 5 feet East of Centerline in order that the results could be correlated with density tests at this location taken during placement of the embankment.

The embankment was excavated with the Trax-Cavator in 10 inch lifts with 3 density tests conducted on each lift. The last few inches of excavation at all levels was done by means of a hand shovel to eliminate possible disturbance due to the loader.

After the densities were taken, the trench was backfilled and compacted with the Trax-Cavator in order to keep the fill intact for further observation.

4.2.4 Settlement Observations

Settlement plates and surface markers were placed at various locations within the fill area as previously described. Readings of initial elevations

and all subsequent elevations were obtained by level readings using a Dietzgen AutoSet Level and a Philadelphia rod. Elevations were referenced to a bench mark located outside the construction area. Readings from the surface markers were obtained by placing the rod on the nail located in each marker. The settlement plate readings were taken by dropping the rod down the access tube to the top of the plates. Rod readings were taken to the nearest 0.01 foot.

4.2.5 Frost Tubes and Thermistors

The frost tubes were effectively self reading since each tube had a scaled tape attached to its side. Readings were obtained by pulling the tube out of the access tubes and noting the level of the change in color solution. This reading was referenced to the surface of the fill; thus the depth of frozen soil was obtained directly.

The soil temperature was obtained frcm each thermistor by connecting them into a calibrated Cole-Parmer Tele-Thermometer. The frost tubes and thermistors were read bi-weekly until the soil was thawed.

V. TEST RESULTS

5.1 Laboratory Results

The soil in the fill was visually divided in two groups. From Station 8+40 south to Station 8+80 the soil was a yellow sand. From Station 8+80 south to Station 9+00 was a gray dirty sand. The discussion of the index properties presented below is the laboratory identification of these two soils.

5.1.1 Grain Size Analysis and Classification

The grain size distribution curves for each of the two soil types were determined by sieve analysis for the material larger than 0.074 mm (#200 sieve) and by hydrometer analysis for the material smaller than 0.074 mm. The average results from 3 sets of sieve analysis and 1 hydrometer analysis are presented in Figure V-1 for the yellow sand. The yellow sand is uniformly graded with approximately 13% of the particles finer than 0.074 mm and 1% of the particles finer than 0.01 mm. The Unified Soil Classification is SM-SW, a border line soil. The results of the gray sand analysis are presented in Figure V-2 with 14% of the particles smaller than 0.074 mm and 2% finer than 0.01 mm. The Unified Soil Classification for this soil is SM. Both soils are marginal for construction purposes.

Atterberg limit tests were conducted on the soil particles passing the #40 sieve for each soil. Both soils exhibit no definable plastic limit. The liquid limit of the yellow sand was 17% and for the gray sand it was 20%. These tests indicate soils of low or zero plasticity.

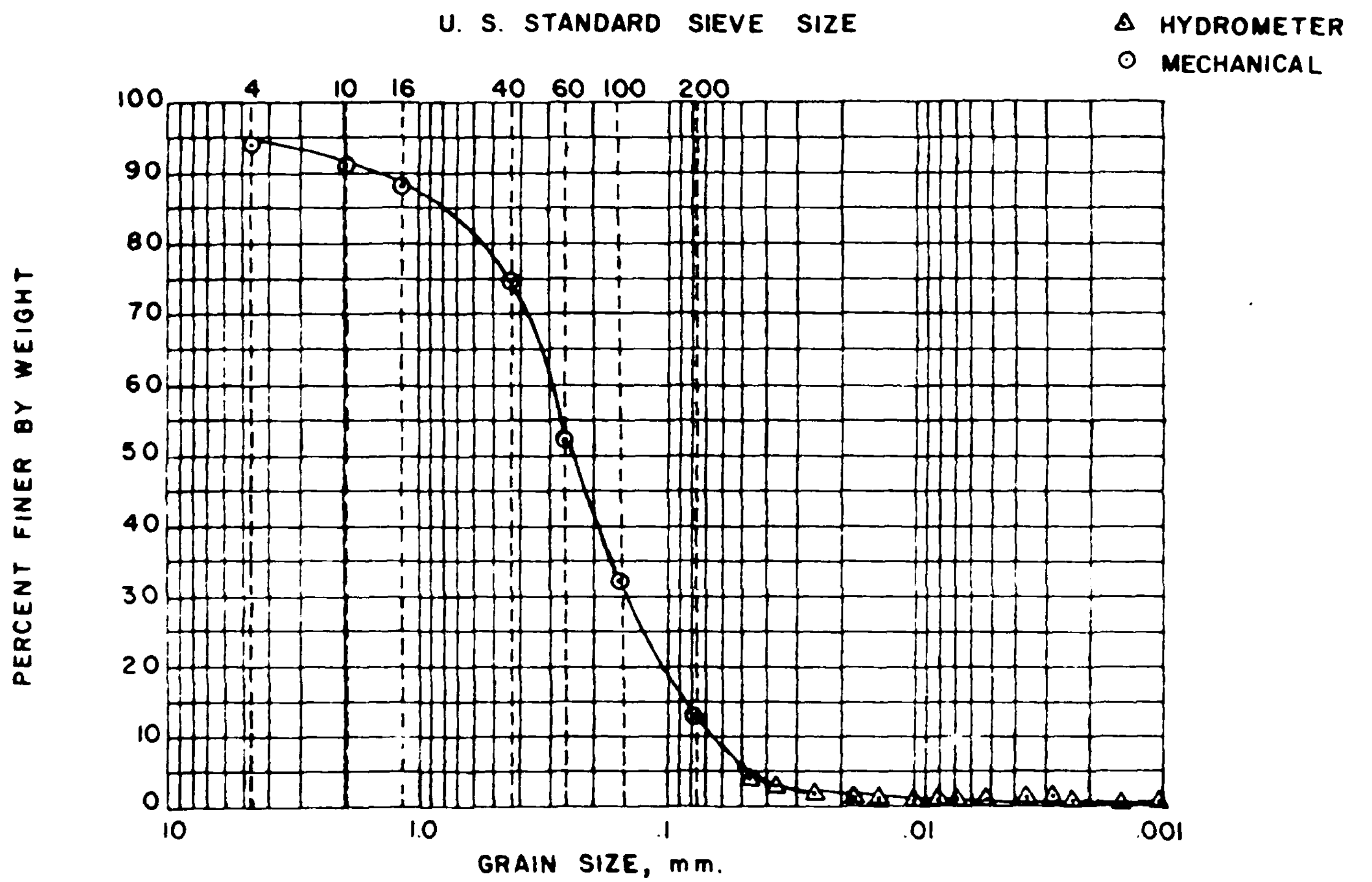

FIGURE V-1 GRADATION RESULTS OF YELLOW SAND, STATION 8+40 - 8+80.

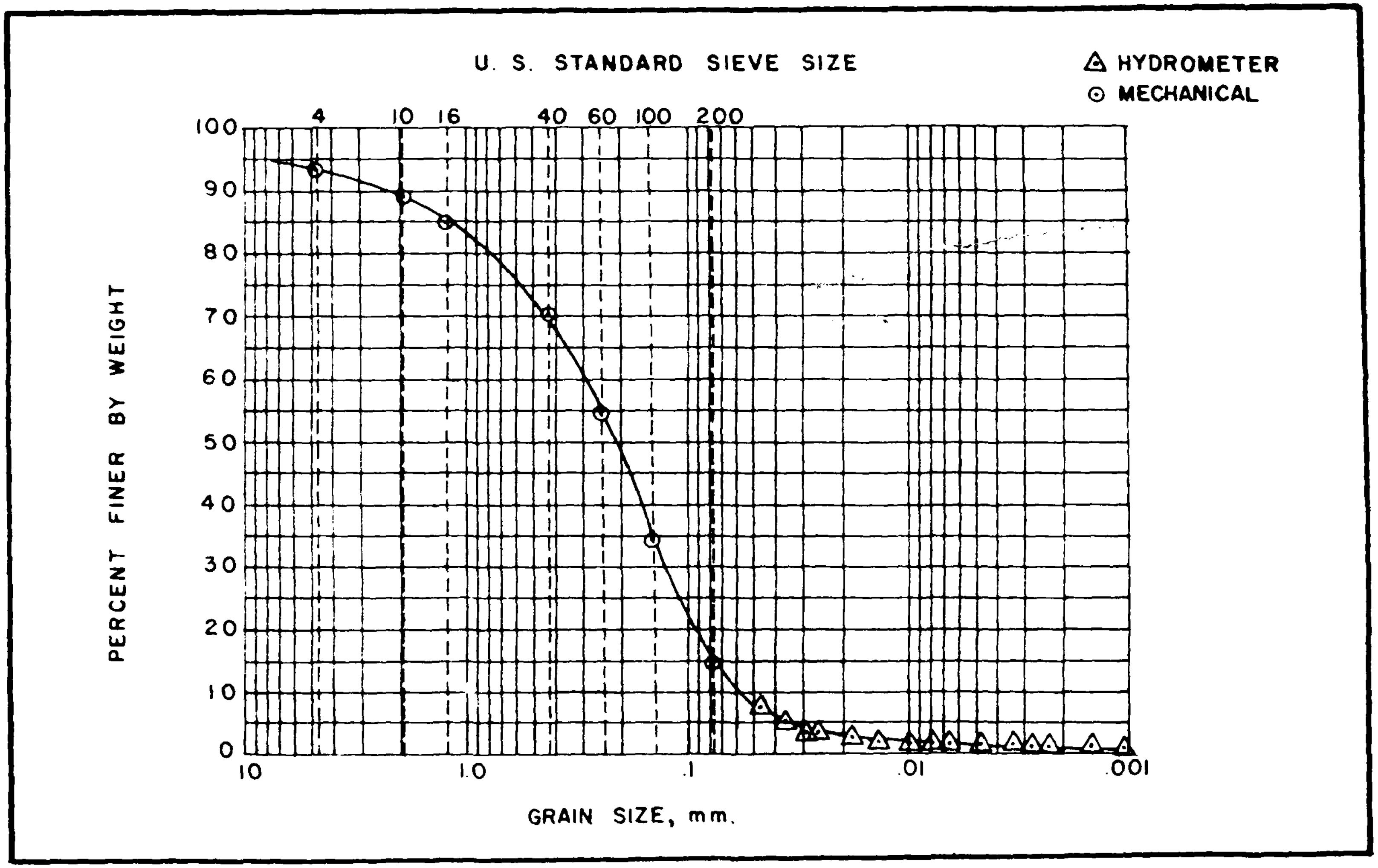

FIGURE V-2 GRADATION RESULTS OF GRAY SAND, STATION 8+80 - 9+00.

5.1.2 Moisture-Density Relationships

The moisture-density relationships were investigated for several different compactive efforts. The results for the yellow sand are shown in Figure V-3. The curves shown in this figure were determined by a least square curve fitting technique. The maximum dry density for Modified AASHO effort is 114 pcf at a water content of 11%. The Standard AASHO compactive effort produced a maximum density of 113 pcf at a water content of 12.5%. The third curve of the set is for the non-standard compactive effort of 5500 ft.-lbs. per cu. ft. A maximum density of 110.5 pcf was obtained for this effort. The three levels of compactive effort show the usual trend of increasing density and decreasing optimum water content as effort increases; however, the magnitude of change is relatively small.

The gray sand was also tested to determine its moisture-density characteristics. The results are presented in Figure V-4. The level of compactive effort used with the soil was the same as for the yellow sand. Modified AASHO effort produced a maximum unit weight of 117.4 pcf at 11% water content. The maximum density for Standard AASHO effort is 112 pcf at 13% moisture content and for the non-standard compactive effort the maximum density was 108 pcf at a moisture content of 13%. The moisture-density relationships for both soils are very similar, with the gray sand being slightly more responsive to increasing compactive effort.

The results of compaction tests conducted on the construction materials when frozen are presented in Figure V-5. Both soil types are included in this figure with little difference noted between the compaction characteristics of the two soils. The results show the usual relationship for compacted frozen soil, with a large decrease in dry density as moisture content is increased.

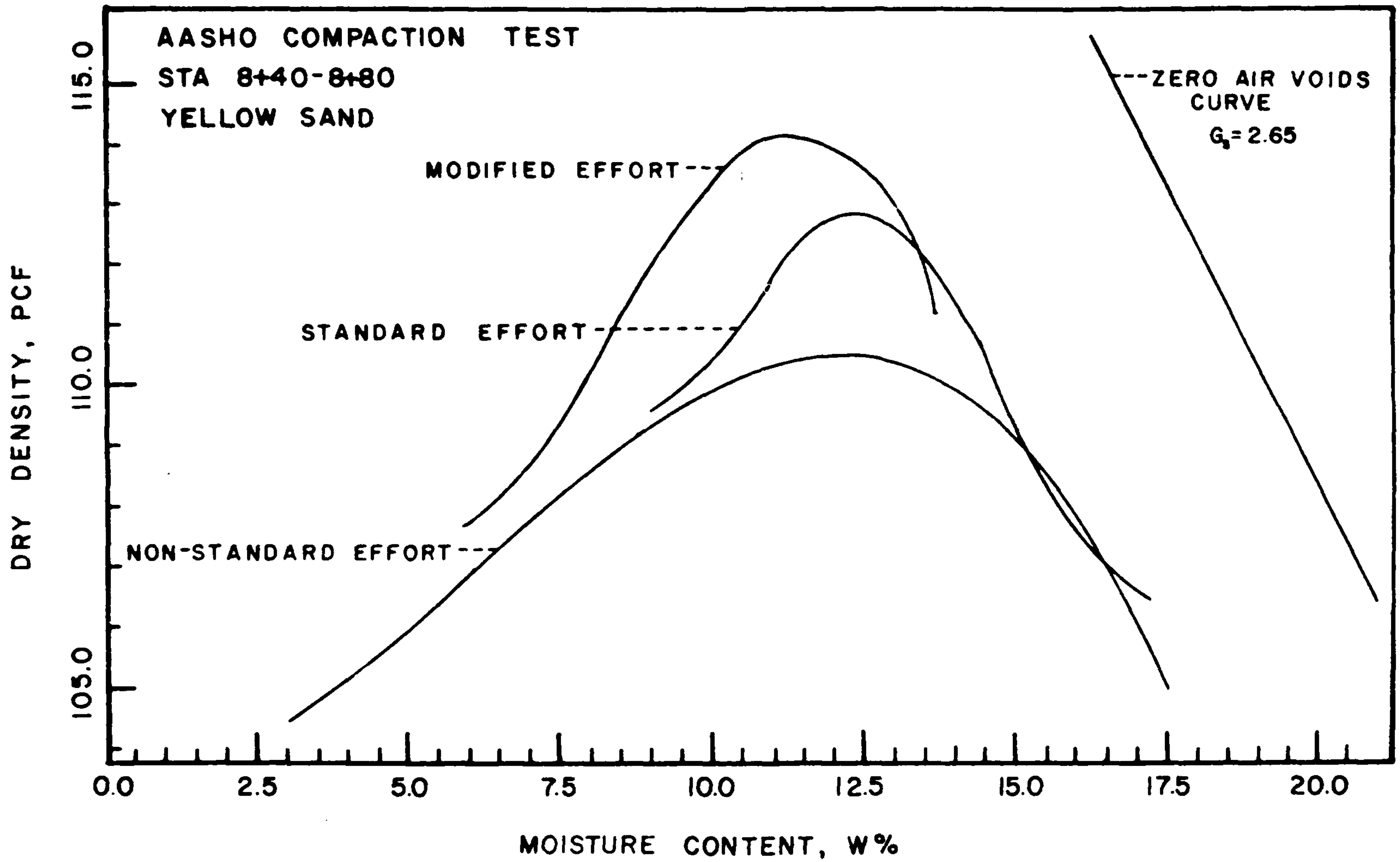

FIGURE V-3 MOISTURE-DENSITY RELATIONSHIP OF YELLOW SAND.

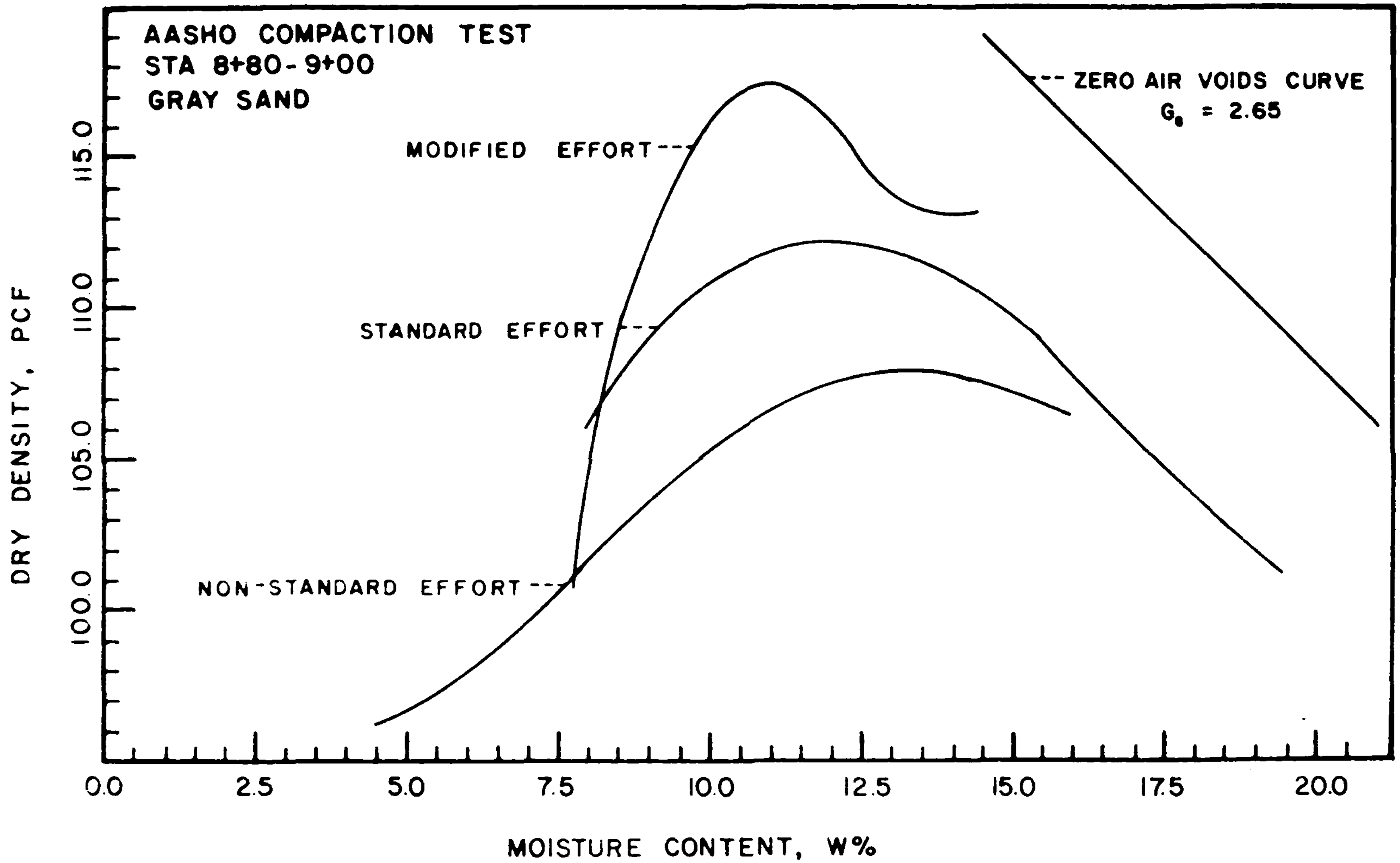

FIGURE V-4 MOISTURE-DENSITY RELATIONSHIP OF GRAY SAND.

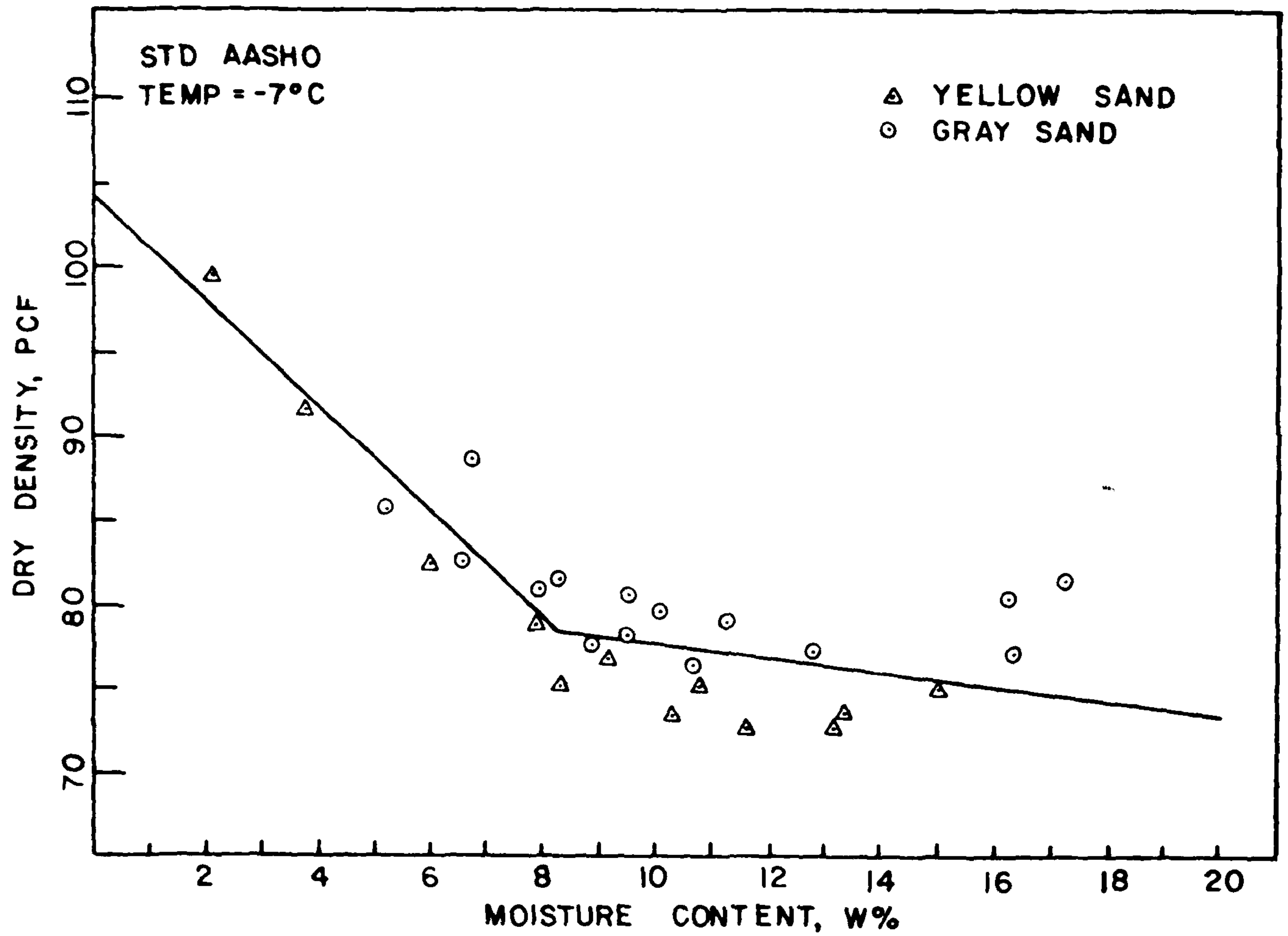

FIGURE V-5 MOISTURE-DENSITY RELATIONSHIP OF FILL MATERIAL AT LOW TEMPERATURE.

5.1.3 Consolidation Results

As a method of determining the compressibility characteristics of the soils, a series of one-dimensional consolidation tests were conducted on the yellow and gray sands. The tests were conducted on samples prepared at moisture contents and densities close to the values obtained in the fill area.

Two different loading sequences were used for the compression testing. One ranged from 0.15 tsf to 20.0 tsf and the other ranged from 0.015 tsf to 1.88 tsf. The compressibility characteristics of the soils at low levels of applied stress can be obtained from Figure V-6. For each soil the sample with the lowest initial density has a much higher compressibility than that of the denser sample as can be determined from the slope of the curves. Also the gray sand, subject to the same levels of stress, showed higher compressibility characteristics than did the yellow sand.

5.2 Borrow Area Test Results

5.2.1 In Situ Densities

Prior to any excavation in the borrow area, in situ densities were obtained using the rubber balloon method. The southern portion of the borrow area was used previously as a haul road during the construction of the Houghton County Airport's runways. High densities within certain parts of the borrow area may be partially due to previous use of these roads.

The sites for the samples were selected arbitrarily but were limited by the ability of the testing personnel to dig a hole in the frozen ground. The results of these tests have been plotted in the form of contours of equal dry density in Figure V-7. The values shown indicate location, dry density and moisture content (in parentheses) for each test taken. The highest densities are located near the intersection of two former haul roads.

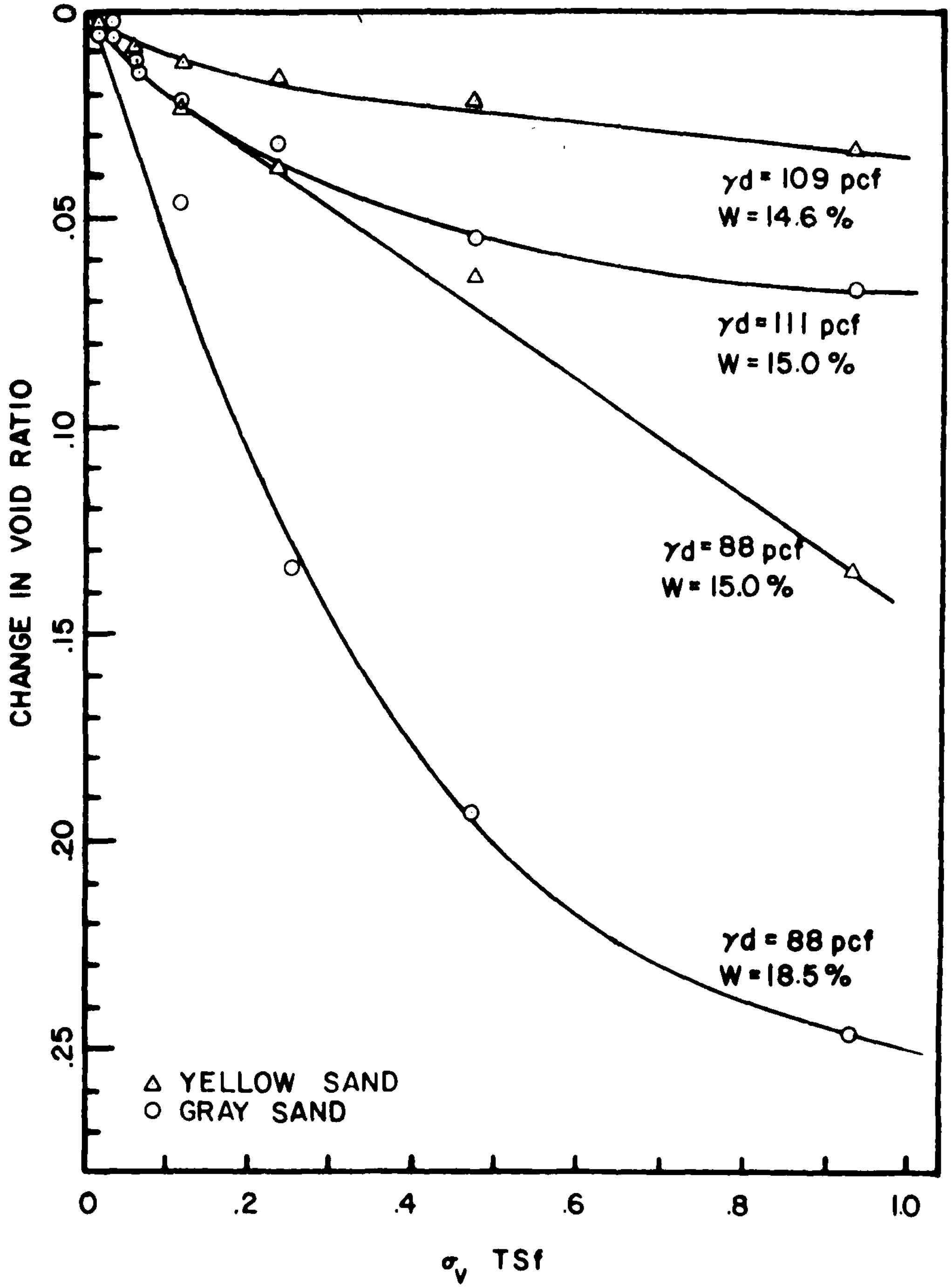

FIGURE V-6 CONSOLIDATION TEST RESULTS OF FILL SOILS.

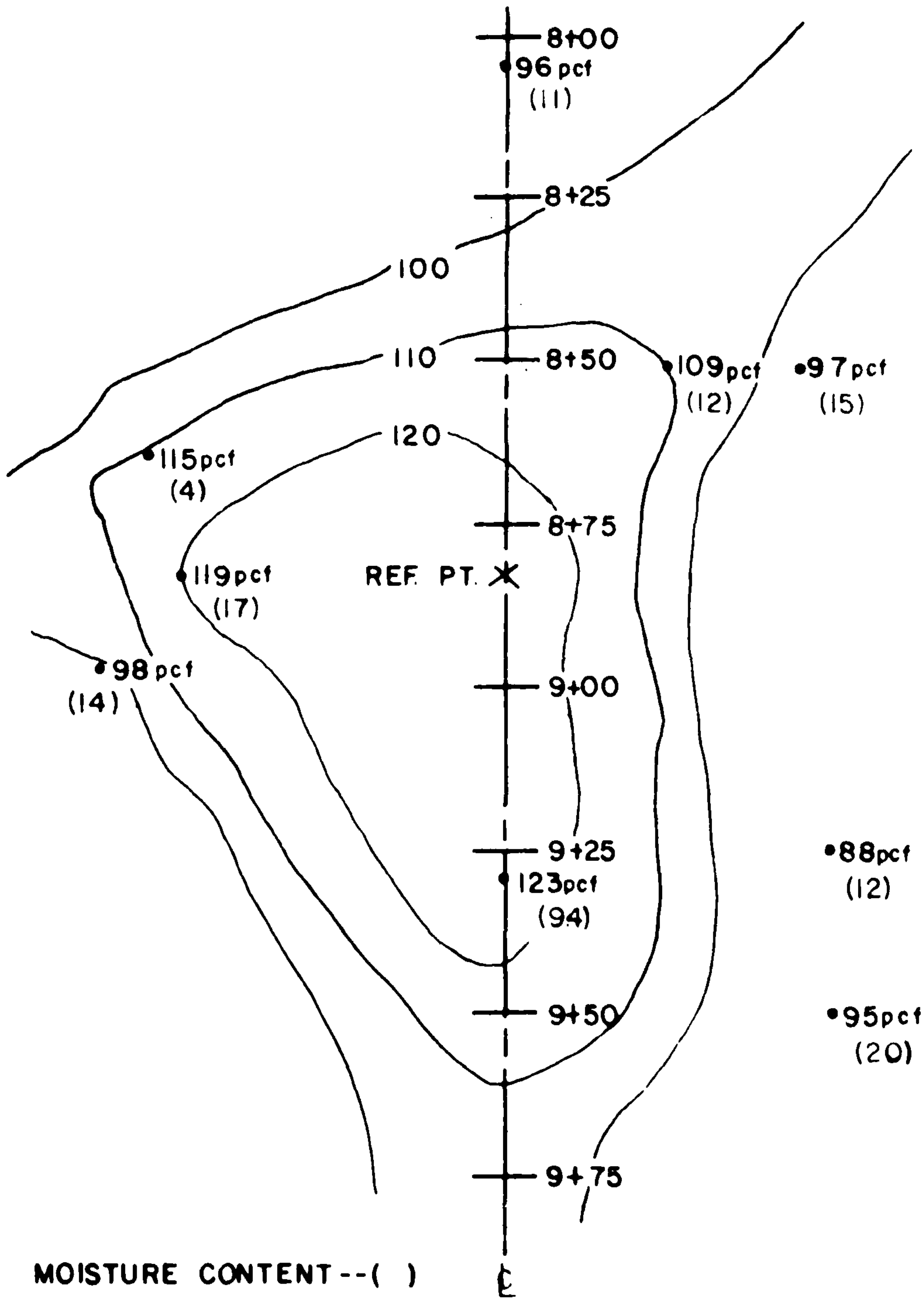

FIGURE V-7 LAYOUT OF INITIAL DENSITIES IN BORROW AREA.

5.2.2 Post Ripping Particle Size Analysis

As part of the test program, the chunks produced in the ripping operation were analyzed by the method previously described. The results from the measuring and sieving analysis are presented graphically in Figure V-8, and in tabular form in Appendix G. This figure shows a fairly consistent distribution of particle sizes regardless of location within the borrow area. There is also a slight tendency of the yellow sand to be more degradable than the gray sand. This was verified by general observation in the field, as most of the large platy chunks were composed of the gray sand.

Further particle size determination was conducted on the stockpiled soils to compare with the tests performed immediately after ripping. Figure V-9 shows the particle size distribution curves for these tests. The gradation curves fall within the envelope established by the post-ripping test. The curves indicate a good distribution of particle sizes with 50% of the material being larger than 6 inches in least dimension. This figure also gives a reasonable estimate of the size of particles in the embankment since the fill material was loaded directly from the stockpile. It is also evident that the stockpiling operation with the D7 had no appreciable effect on the particle size distribution. Therefore, it can also be assumed that the loading and placing operation did not degrade the particles by any appreciable amount.

5.3 Fill Site Test Results

5.3.1 Initial Densities

In order to insure the existence of good foundation soil for the embankment, densities were taken throughout the fill site. The densities were taken just outside of the immediate embankment plot where the snow had not been removed and frost had not penetrated. The auger borings taken

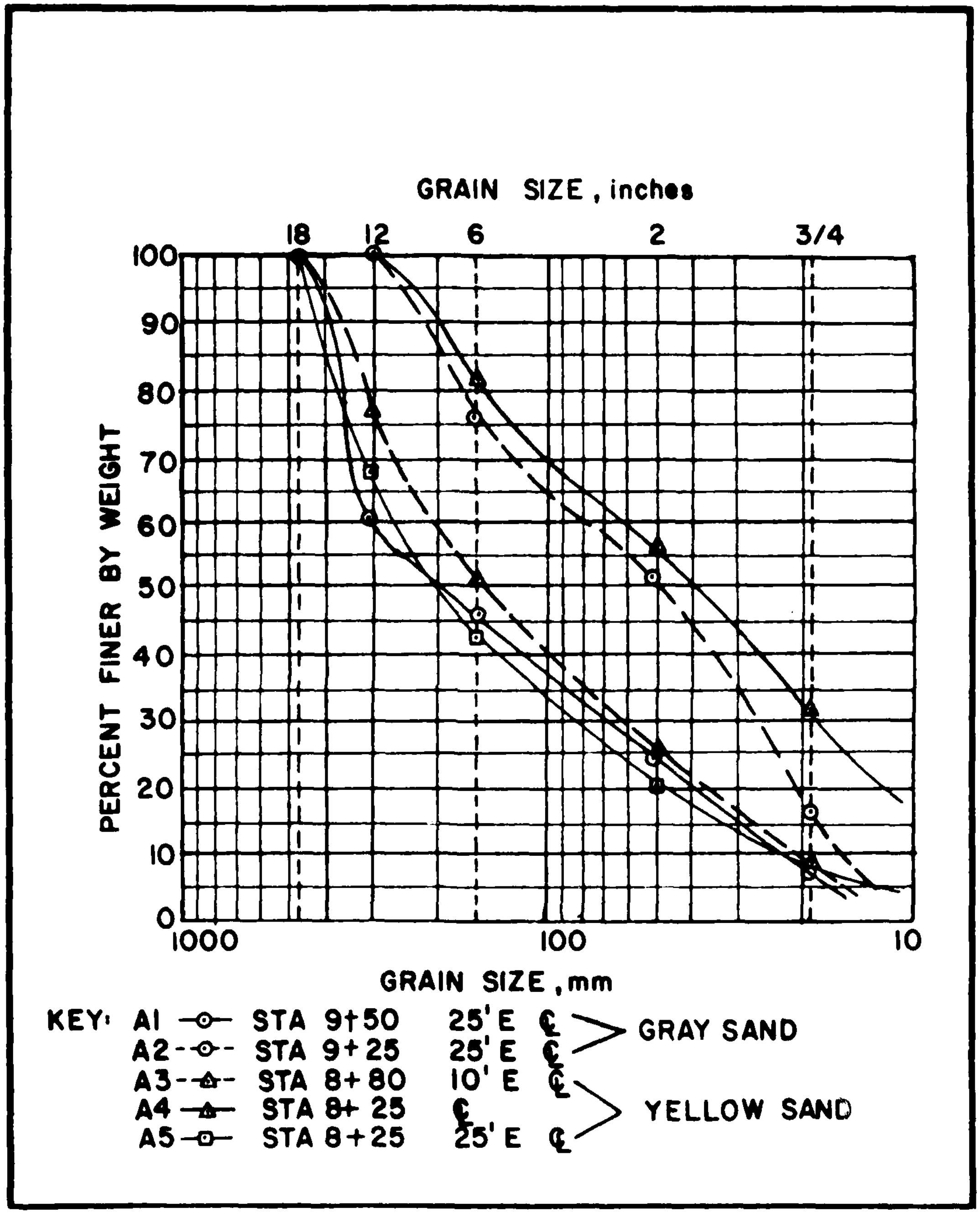

FIGURE V-8 FROZEN CHUNK GRADATION CURVES.

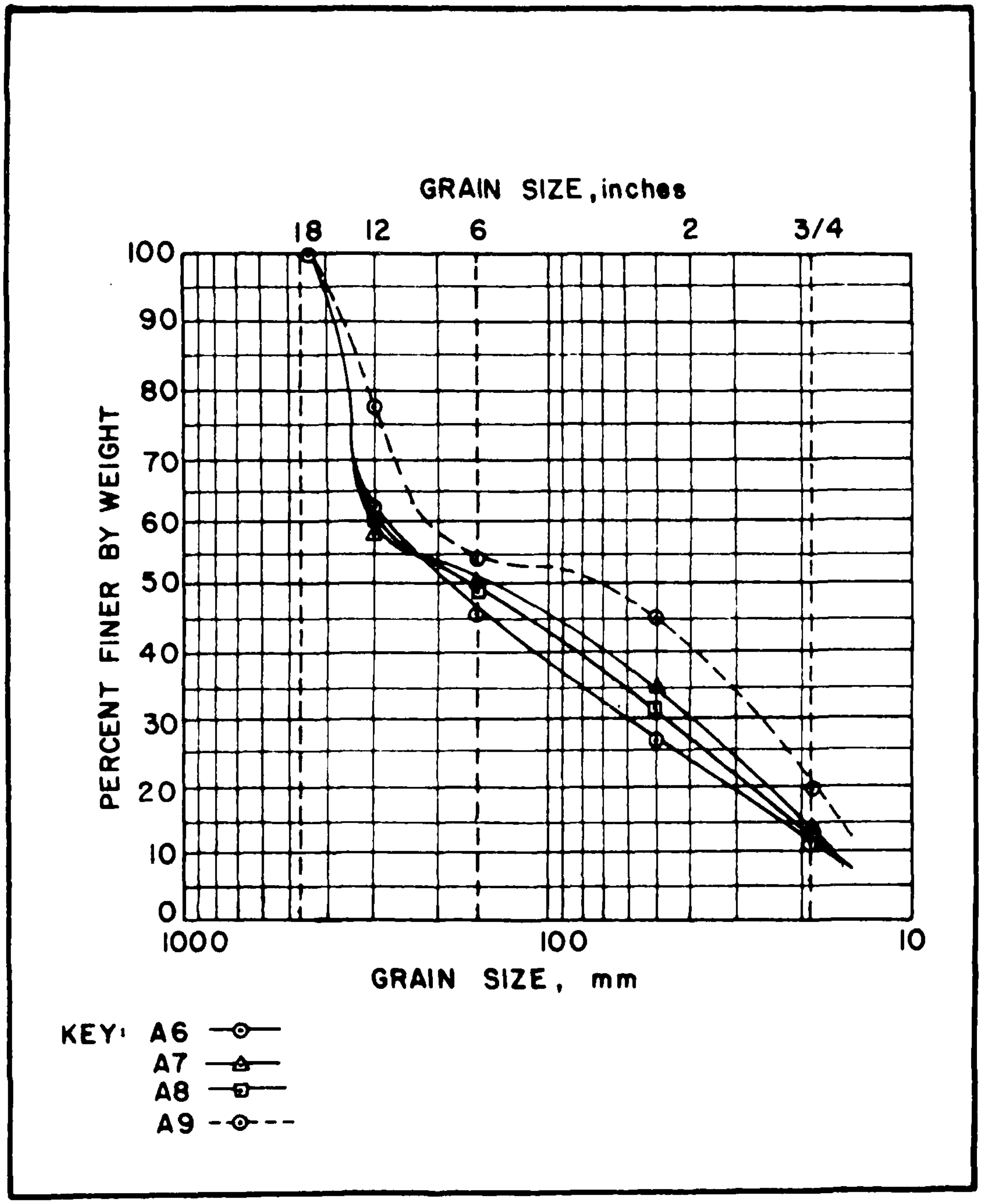

FIGURE V-9 FROZEN CHUNK GRADATION CURVES OF STOCKPILED BORROW MATERIAL.

prior to the snow removal indicated a uniform soil south of Station 8+20. Therefore it was assumed that the densities obtained were representative of the foundation soil. The dry densities ranged from about 90 pcf to 110 pcf with moisture contents of 9 to 21 percent. It was believed that the sandy foundation soil was more than adequate to carry the relatively small embankment. Therefore, no attempt was made to compact the foundation soil prior to placement of the fill.

Foundation soil densities are tabulated in Appendix B under "Fill Area Field Densities."

5.3.2 Embankment Densities

The efficiency of the embankment compaction process was determined by density tests taken at predetermined stages of compaction. Density tests were taken at Stations 8+60 and 9+00 before compaction, after 4 passes and after 10 passes with the sheepsfoot roller on each lift. The dry densities and moisture content obtained from the field tests are presented in Table V-1. In this table the combined averages include the tests for both soils for a given number of passes on each lift. The total embankment average is the level of compaction achieved for the entire embankment using a given number of passes. The total embankment average is presented for both individual soils and for the soils combined.

The average value of dry density for the entire embankment of 78 pcf prior to compaction; 100 pcf after 4 passes and 96 pcf after 10 passes. The total embankment averages indicate the gray sand is slightly more compressible than the yellow sand in the frozen state which was also true of the soils in the unfrozen state as shown in the lab compaction data.

When the embankment had completely thawed, in situ density tests were conducted to determine the change in density upon consolidation of the embankment. For these tests 10 inch layers of soil were removed and density tests

TABLE V-1 FIELD COMPACTION RESULTS
Dry Density, pcf.

LAYER	Yellow Sand Sta. 8+60			Gray Sand Sta. 9+00			Combined Average		
	NUMBER OF PASSES			NUMBER OF PASSES			NUMBER OF PASSES		
	0	4	10	0	4	10	0	4	10
1 (Bottom)	63.7 (23.3)	95.0 (15.6)	104.0 (17.3)	60.0 (16.3)	102.7 (14.1)	105.9 (17.1)	61.9 (19.8)	98.9 (14.9)	105.0 (17.2)
2	80.1 (21.2)	89.9 (15.3)	83.1 (19.3)	84.6 (16.9)		104.6 (17.6)	82.4 (19.0)	89.9 (16.6)	93.9 (18.4)
3	82.5 (19.5)	72.5 (25.5)	80.5 (19.7)	81.2 (20.2)	97.4 (12.8)	82.2 (16.7)	81.8 (19.9)	85.0 (19.2)	81.4 (18.2)
4	113.8 (13.3)	108.9 (10.8)	94.1 (10.4)	89.3 (15.2)	126.2 (13.9)	86.1 (13.9)	101.6 (14.3)	117.6 (13.2)	90.1 (12.2)
5 (Top)	70.0 (13.5)	107.4 (14.5)	102.8 (17.1)	48.3 (24.0)		120.0 (14.4)	59.2 (18.8)	107.4 (14.5)	111.4 (15.8)
Total Embankment Average	78.0 (18.8)	94.7 (16.3)	92.8 (16.8)	75.9 (17.1)	108.8 (15.1)	99.8 (15.9)	77.4 (18.4)	99.8 (15.7)	96.4 (16.4)

(   ) Moisture Content

conducted at each level. The average dry densities and moisture contents are presented in Table V-2. The dry densities near the

TABLE V-2 POST CONSOLIDATION DENSITY TEST RESULTS
Sta. 8+60, July 15,1975.

ELEVATION, feet.	AVERAGE DRY DENSITY, pcf.
99.9 (Surface)	113.0 (4.0)
99.0	113.5 (7.6)
98.2	114.8 (8.4)
97.4	114.1 (13.1)
96.6	103.1 (13.6)
95.8 (Bottom of Embankment) (Foundation Soil)	109.5 (14.0)
95.0	128.9 (14.4)

(   ) Moisture Content, %

surface are higher compared to the values near the bottom of the embankment due to the increase in moisture content in the lower levels of the structure. The density of the bottom layer is very nearly the same as the density obtained just after compaction while the soil was still frozen (104 pcf, Table V-1). However, the dry density generally increased approximately 10% in the upper part of the fill.

5.3.3 Frost Line and Thaw Profile

Soil temperature and depth of frost within the embankment was monitored using frost tubes and thermistors. Initial readings of the thermistors just after installation (March 19, 1975) showed ground temperatures were slightly above freezing (33 - 35°F). However, by March 21 the backfill material had reached thermal equilibrium with the embankment material and the temperature readings dropped below 32°F throughout the fill and into the subgrade soil. The fill temperature continued to drop until it reached a minimum (26.9°F) at a depth of 1 ft. on April 2, 1975. A series of typical temperature profiles taken on various dates are presented in Figure V-10, and soil temperatures are tabulated in Appendices E and F. The temperature logs show a large fluctuation in temperature in the upper 3 ft. of the embankment with the bottom foot of fill and the foundation soil changing slowly from slightly below 32°F to 40°F. It can also be observed from Figure V-10 that the fill thawed from only the top and the thawing action from the bottom took place only in the foundation soil.

The depth of frost was also obtained from the frost tubes. These instruments cannot be used to obtain ground temperatures directly but are a good supplement to the thermistors for obtaining the location of the frost line. In general the depth to the 32°F isotherm from the top of the fill could be located by either the frost tubes or the thermistors with good agreement. However, the bottom 32°F isotherm located by the thermistors was lower than indicated by the frost tubes. This was due to the low

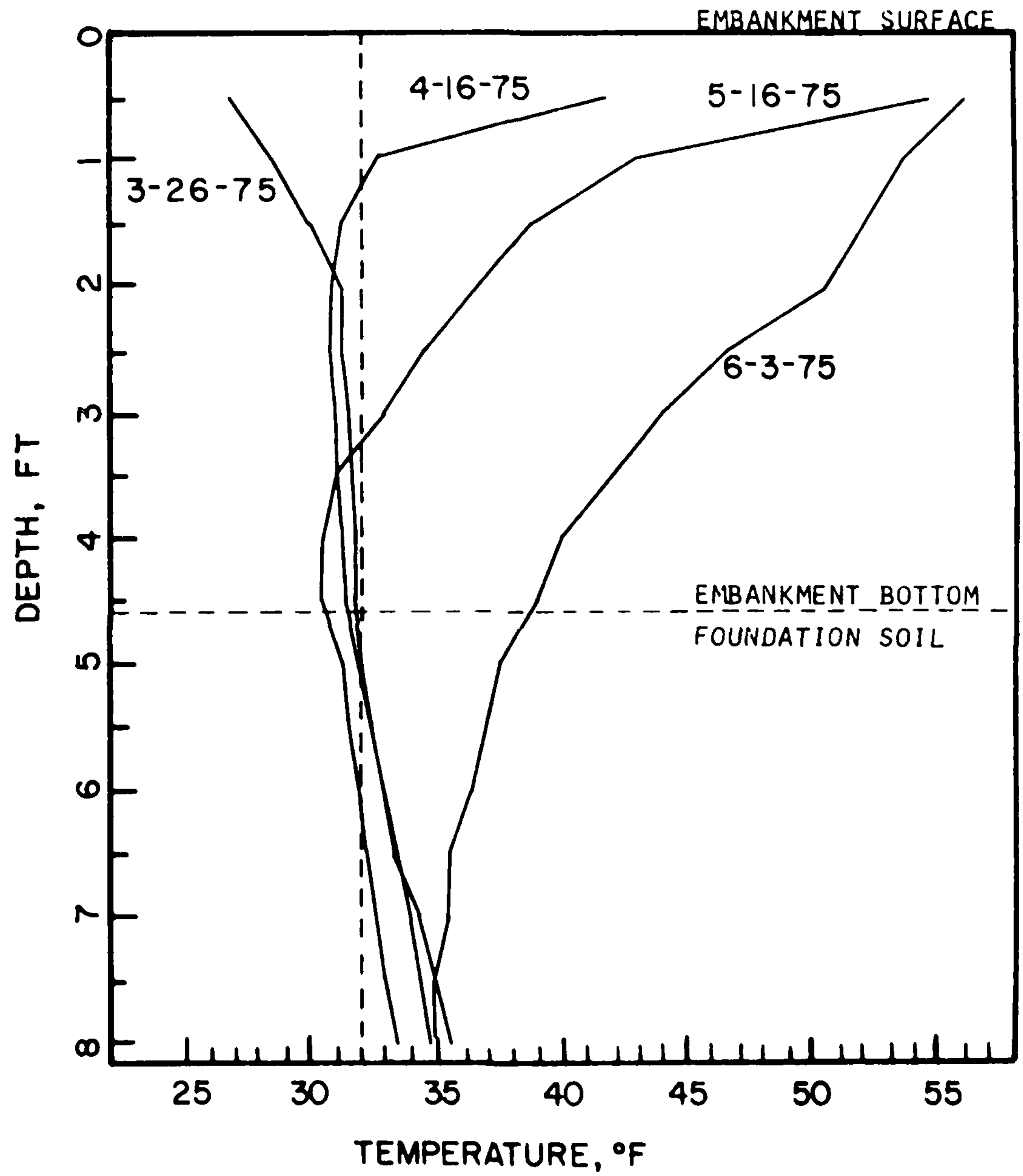

FIGURE V-10 TEMPERATURE PROFILES IN EMBANKMENT AT STATION 8+70.

temperature gradient near the base of the fill and to the insulating effect of the air around the frost tubes.

Time dependent changes in temperature within the fill were provided by periodic readings taken with the thermistors and frost tubes. Since the lower $32^{\circ}F$ isotherm was within the foundation soil, the top $32^{\circ}F$ isotherm indicated the depth of thaw that took place within the fill. A plot of depth of thaw versus time at two locations in the embankment is shown in Figure V-11. Also shown on this figure is the average daily temperature to show the response of the frost line to the daily temperatures.

Figure V-11 shows a high rate of thaw soon after the instrumentation was installed. However, rate of thaw was slowed by a drop in the daily temperature. In the first week of May the rate of thaw again increased at a near linear rate until the fill had completely thawed.

The two depths of thaw curves on Figure V-11 show the effect of the location within the fill on the rate of thaw. Station 9+00 was located on the south end of the fill which was subject to more surface radiation and thawing from the top, the sides and from the south end. Station 8+70 was located in the center of the fill and was thus more protected and not subject to thawing in the direction parallel to the embankment.

5.3.4 Settlement Observations

The development of settlement within the embankment area was highly dependent on time and the degree to which the frozen embankment material thawed. Typical plots of settlement versus time for the centerline at Station 8+70 and Station 9+00 are presented in Figures V-12 and V-13 respectively. Also shown is the settlement of the first layer settlement plates. No significant settlement of the surface markers occurred until April 15 at either station. From this date, there was a steady increase in total settlement until the 1st of June, followed by some residual

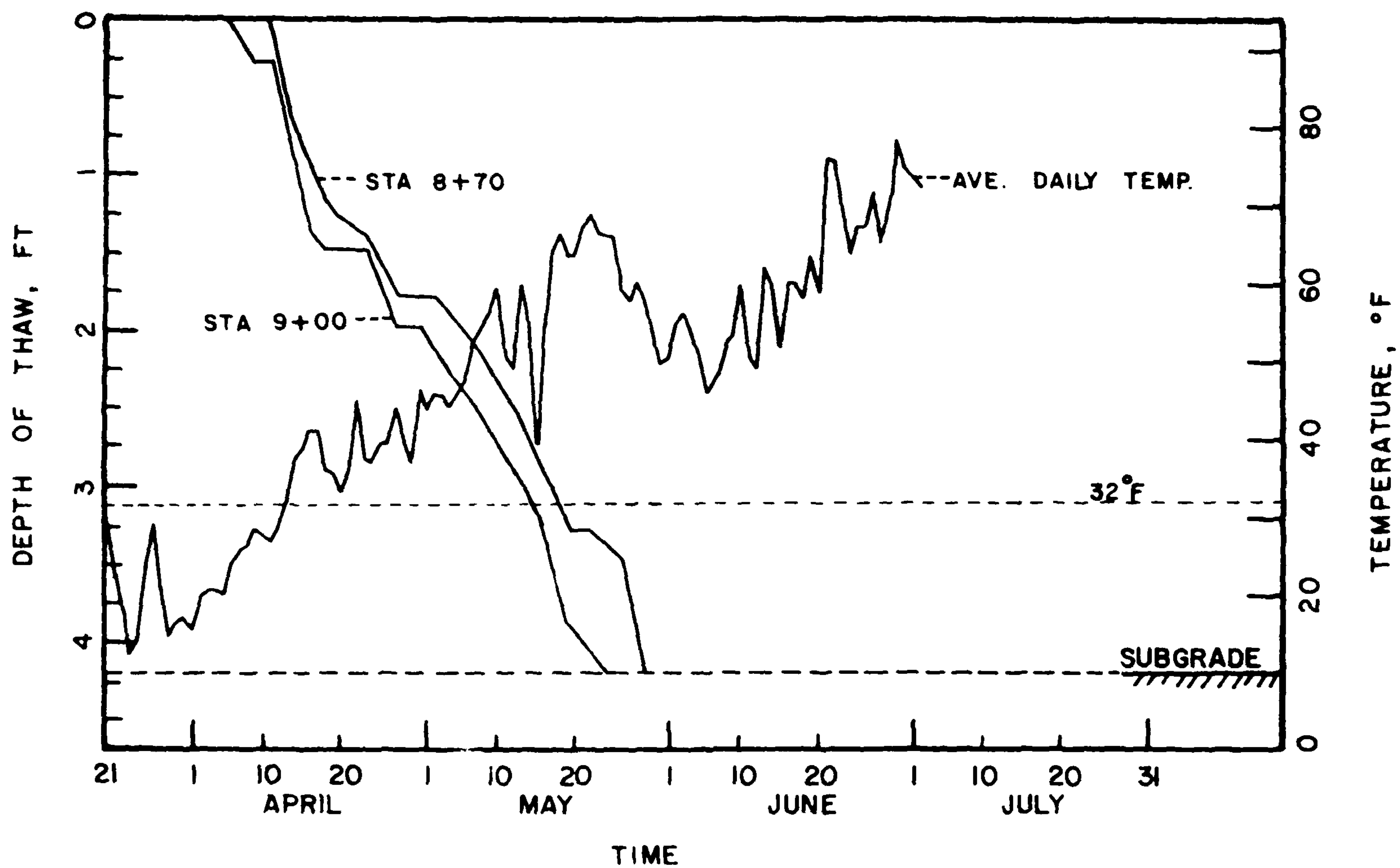

FIGURE V-11 AVERACE DAILY AIR TEMPERATURE AND DEPTH OF THAW VERSUS TIME.

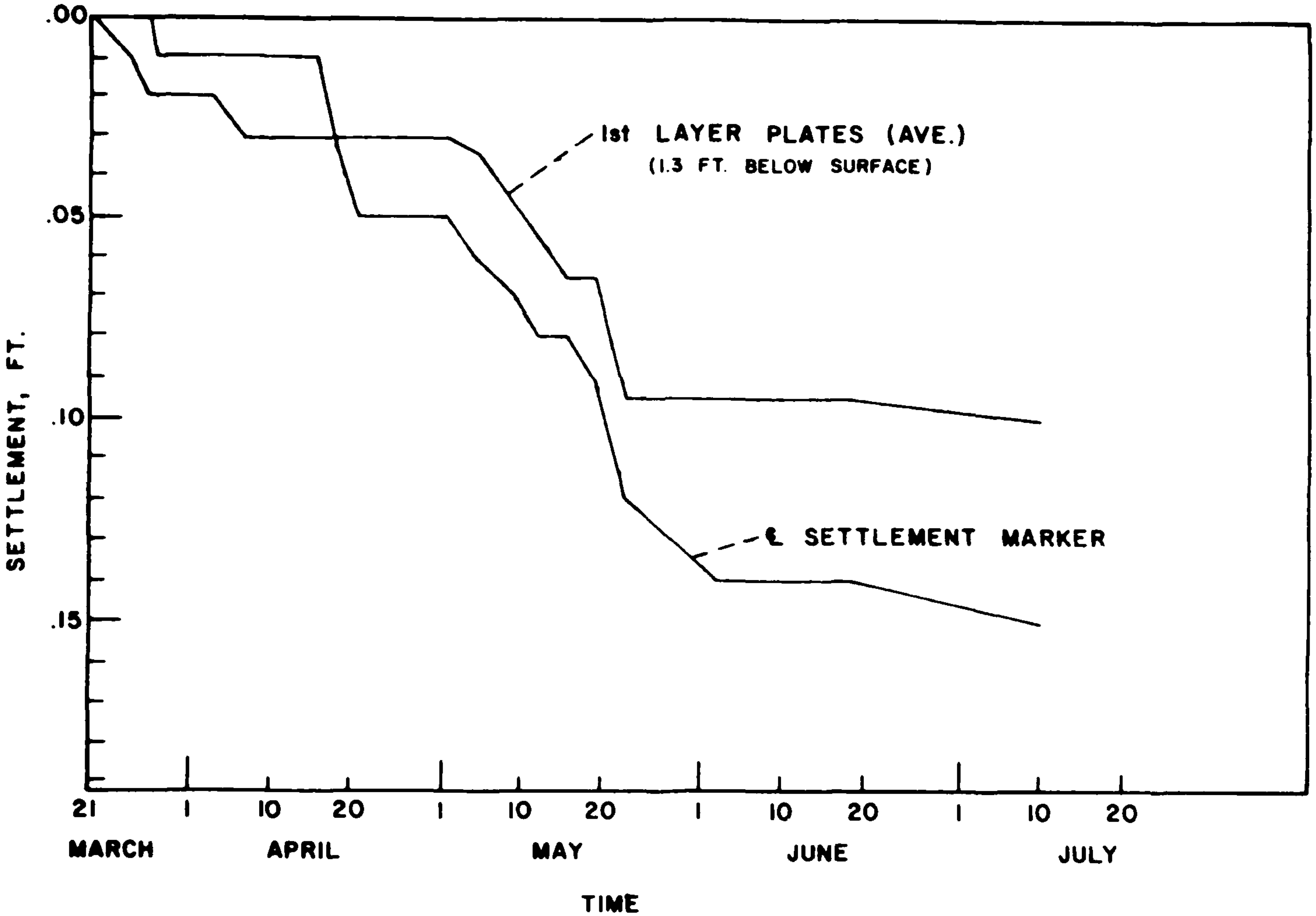

FIGURE V-12 MEASURED SETTLEMENT AT STATION 8+70

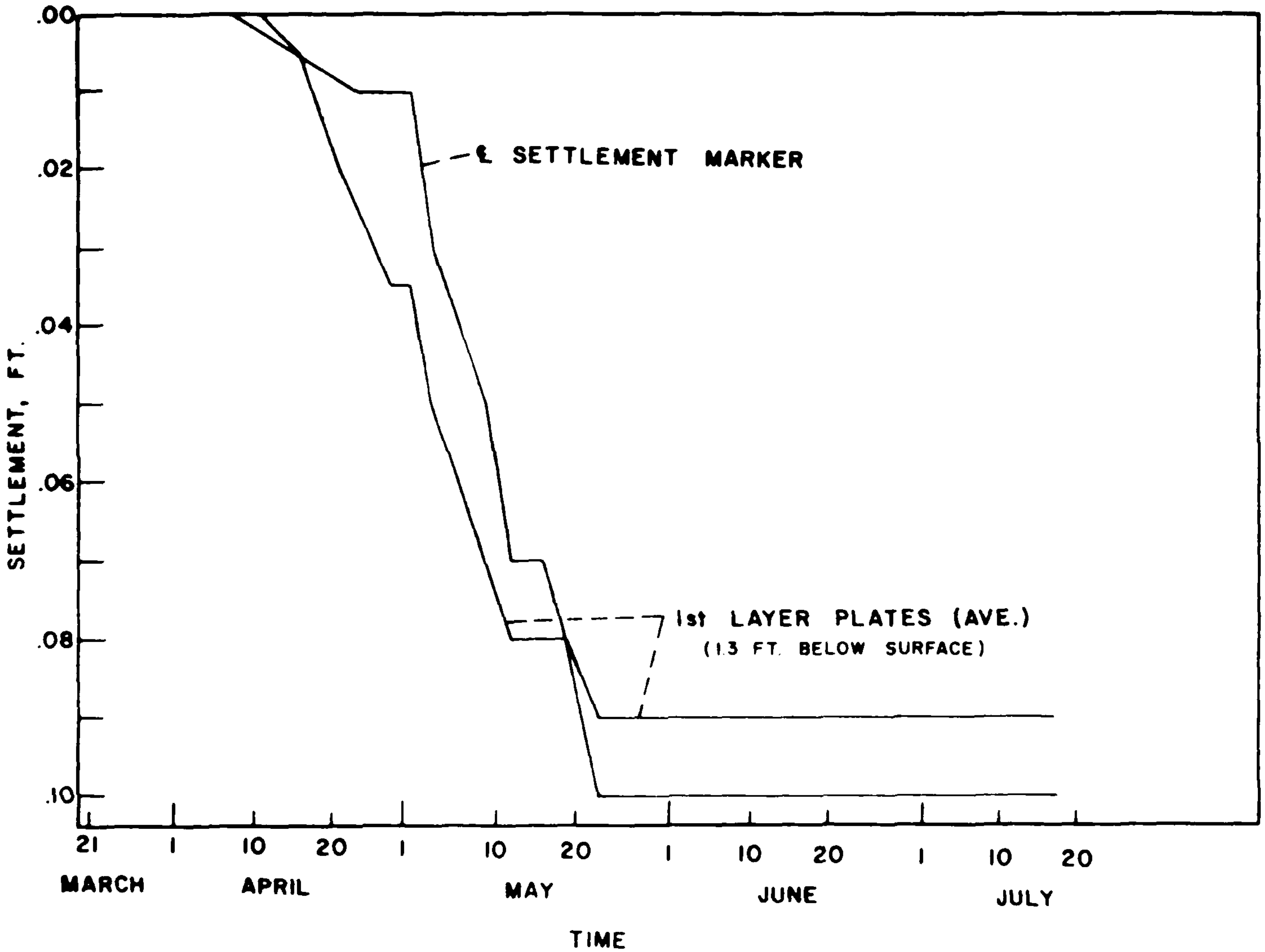

FIGURE V-13 MEASURED SETTLEMENT AT STATION 9+00

settlement. Other surface markers showed similar results with most of the settlement occurring between April 15 and June 4.

The settlement plates generally lagged behind the surface markers in total settlement as can be seen from Figure V-12. This is due to the direction of thaw in the fill from top to bottom. This was not true of the settlement plates at Station 9+00 due in part to the increased susceptibility of the end of the fill to thaw as well as the skewed orientation of the fill which decreased the cover on some of the settlement plates. The decreased cover increased the thawing over the plates but did not affect the centerline marker. Thus, the settlement plates at Station 9+00 tended to run slightly ahead of the centerline surface marker in the rate of settlement.

As might be expected the embankment settlement was not constant throughout the fill area. Total settlement ranged from 0.11 to 0.19 foot along the west edge of the top of the fill. Figure V-14 shows contours of equal settlement for the entire embankment area. The largest total settlement (up to June 4) was in the northwest quadrant of the embankment which is also the area of the highest amount of fill. Due to the different heights of fill material, no direct correlation could be made with settlement and soil type. Settlement data are tabulated in Appendices C and D.

The magnitude of settlement within the foundation soil was obtained from readings taken to the top of settlement plates placed at the base of the embankment. Observed settlements for the period from March 19 to July 15, 1975 are tabulated in Table V-3. The settlement varied with location in the fill and was largest in the north and west sides of the fill.

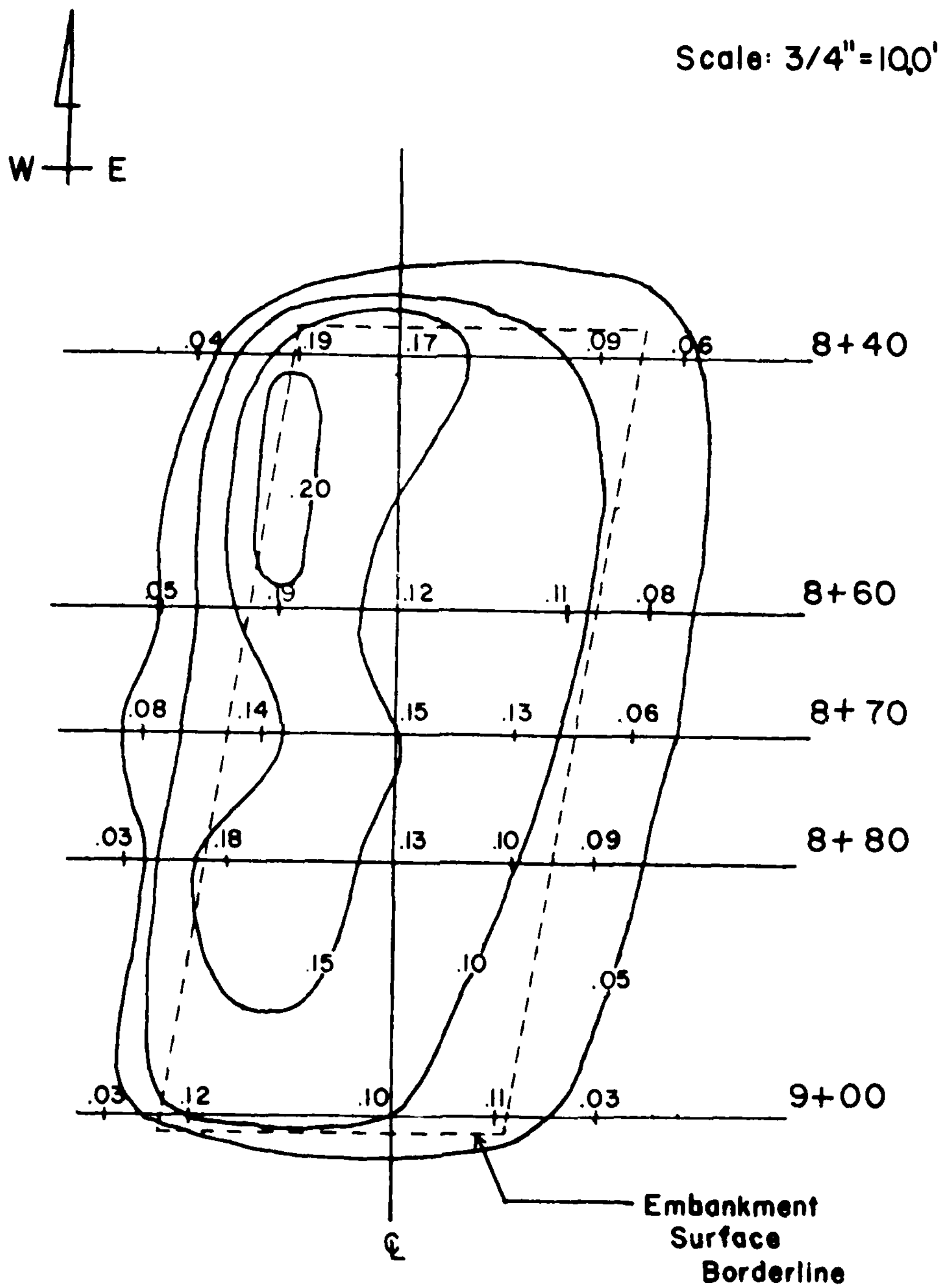

FIGURE V-14 CONTOURS OF EQUAL SURFACE SETTLEMENT IN THE EMBANKMENT.

TABLE V-3. OBSERVED SETTLEMENT OF FOUNDATION SOIL*

Station	Settlement, feet		
	10 ft. West of CL	10 ft. East of CL	Average
8+40	0.04	0.08	0.06
8+70	0.11	0.03	0.07
9+00	0.04	0.00	0.02
		Average	0.05

*March 25 - July 15, 1975

The average foundation settlement of 0.05 foot can only be assumed accurate in the main body of the embankment where the data were collected. The settlement of the foundation soil beneath the side slopes of the embankment was of smaller magnitude because this area was not as highly stressed as the subgrade soil beneath the main body of the fill.

VI. DISCUSSION OF RESULTS

6.1 Ripping

In a comprehensive study of winter earthwork, Yoakum (42) concluded that tractor mounted rippers can efficiently loosen frozen soil and stated, "the spacing and number of passes by the tractor-ripper should vary with conditions of the frozen soil and desired breakage size." However, when ripping soils for this project, it was impossible to follow any predetermined plan for spacing or number of passes and still rip the soil in an efficient manner. This was due to several factors including: 1) equipment limitations, 2) variable frost depth, and 3) high moisture content in the soil.

The ripping operation for this project commenced with an initial pass of the dozer through the borrow area. Typically, the initial pass of the dozer failed to penetrate the frozen ground. But by working over several areas, a spot was found where the ripping tooth could penetrate the ground and start breaking the frozen crust. Once an initial penetration was made in an area, the D7 worked that area until it was completely broken up. There did not appear to be any particular spacing between passes that would uniformly break up the frozen soil, and the process eventually became one of working over an area in any direction that would allow the D7 to get through the frozen soil.

During the ripping there was a natural tendency to drive the ripping tooth to its full depth (12 in.). At this depth the D7 could not move forward and it was necessary to extract the tooth to about 6 inches before it could move forward. The operator had to continually adjust for this

tendency of the ripping tooth to penetrate to a depth beyond the capability of the tractor. It was evident that a larger bulldozer with a conventional mounted ripper tooth would have been more efficient in loosening the frozen crust.

The initial ripping produced chunks in the range of 4 to 6 feet maximum diameter and 2 feet thick. The thickness was, in general, an indication of the depth of frozen soil. Near Station 8+80 (the area of lowest snow accumulation) the soil could not be broken up, but in the area to the north and south of this point the soil was eventually ripped up enough to be stockpiled for placement in the fill. In many instances, it was possible to reduce the size of the frozen particles by driving the dozer over the particles and crushing them with the weight of the dozer.

There seems to be, as expected, a relationship between the soil and the ease with which it could be ripped. The yellow sandy soil to the northwest of Station 8+80 was much easier to rip than the dark sandy soil to the southeast of Station 8+80. This was due, at least in part, to the higher moisture content and in situ density in the area to the southeast.

Using the procedure described above, ripping produced a wide range of sizes, with particles larger than 3 feet being quite common. The actual gradation curves (Figures V-8 and V-9) of the chunks produced by the tractor ripper reveal an even distribution of chunk sizes with 5 to 15% of the particles being smaller than 10 mm. However, the unfrozen soil (Figures V-1 and V-2) had approximately 95% of the particles smaller than 10 mm. This is a vivid example of the agglomerating effect of the frozen pore fluid.

As noted earlier, the dirty gray sand had generally higher frozen densities than the clean yellow sand. However, the variation in density did not appear to affect the gradation of the chunk size since both

materials produced similar gradation curves.

6.2 Compaction

It is generally observed (11,17,5) that for comparable compactive efforts a frozen soil will have a lower dry density at a given water content than the same soil in the unfrozen state. The difference in dry density between the unfrozen and frozen soil is a function of several factors such as moisture content, gradation, soil type and method of compaction. Of these factors water content is most important, particularly at high percentages.

One of the primary objectives of the tests conducted as part of this project was to determine the effectiveness of field compaction of frozen soils by relating the field density to appropriate laboratory tests conducted on the same soil. Because of the variable moisture content in the field soils and the variation in temperature during compaction, it is not possible to conclusively relate the densities achieved in the field to those obtained by laboratory cold-room tests. However, some observations may be made which provide some insight to the question. The following discussion is based on an interpretation of the data based on trends which might be expected, and on further evaluation of a previous laboratory cold-room compaction study (11).

As previously stated, field density tests were taken after spreading the frozen soil on the fill, but before compaction. This was identified as "zero" passes. However, it should be recognized that the action of spreading with a bulldozer does provide some compaction, however minimal. In spite of this, one would expect a considerable variation in the density resulting from spreading only. This is shown in Figure VI-1a, a plot of dry density as a function of moisture content for both soils. Also shown is the zero air-voids curve for a frozen soil with a specific gravity of 2.65, the conventional compaction curve, and the lab curve for frozen soil. As might be expected,

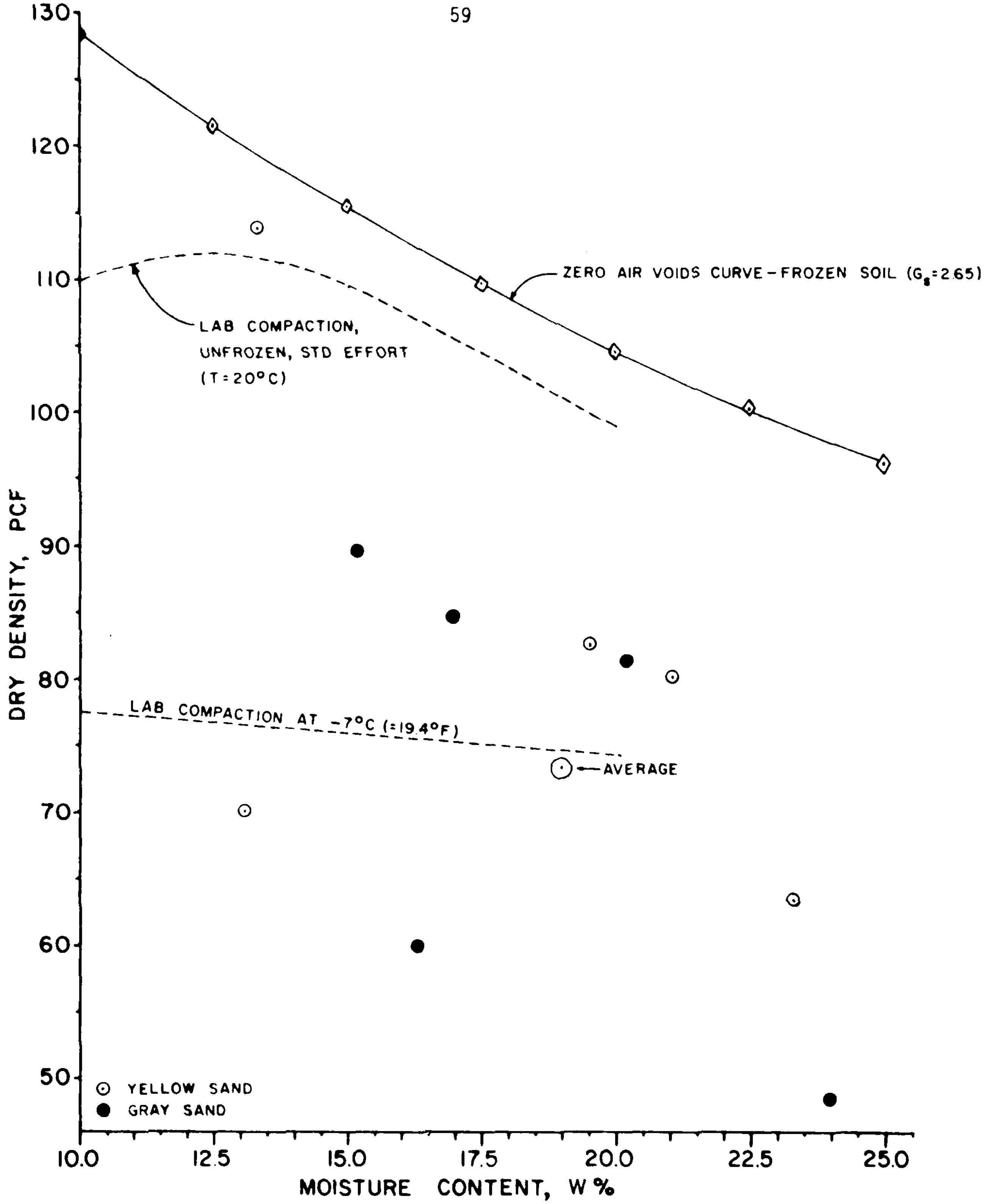

FIGURE VI-1A MOISTURE DENSITY RELATIONSHIPS - ZERO PASSES OF ROLLER COMPARED TO LABORATORY COMPACTION

there is considerable scatter. This is reasonable, as one of the objectives of compaction is to make the soil mass more uniform in density.

Greater uniformity is in fact shown in Figure VI-1b, a plot of the fill densities after four passes of the roller. Except for the point at 25 percent moisture, the pattern of points definitely has less scatter. As will be discussed later, the spread along a line approximately parallel to the zero air-voids curve may not be scatter at all, but a reasonable variation of density with water content. The scatter away from any such parallel line may be the effect of various compaction temperatures. It will be noted that one point falls above the zero air-voids curve, and hence may be suspect.

The results after 10 passes are shown in Figure VI-1c. Overall, this plot does not show a closer pattern than the one for four passes. Again, one point is above the zero air-voids curve.

The scatter of the data may be due to several factors, including variations in texture, variations in density of the soil chunks before ripping from their original position, the difficulties in obtaining accurate field density samples, and variations in the temperature of the soil when compacted. For the purpose of analyzing the effect of number of passes, these items will be accounted for by using only the average density and water content for each of the three levels of compaction. Thus the assumption is made that each of the three sets of data are affected by the several factors stated above in about the same manner. However, points plotting above the zero air voids curve were not included in the average, nor was the point at 113.8 pcf in the zero-pass group.

These calculated average densities and water contents are plotted on their respective figures, and also on Figure VI-1d. The latter figure also includes the zero air-voids curve, the lab compaction curve for the unfrozen soil, the pertinent portion of the frozen compaction curve, and the extension of the initial portion of

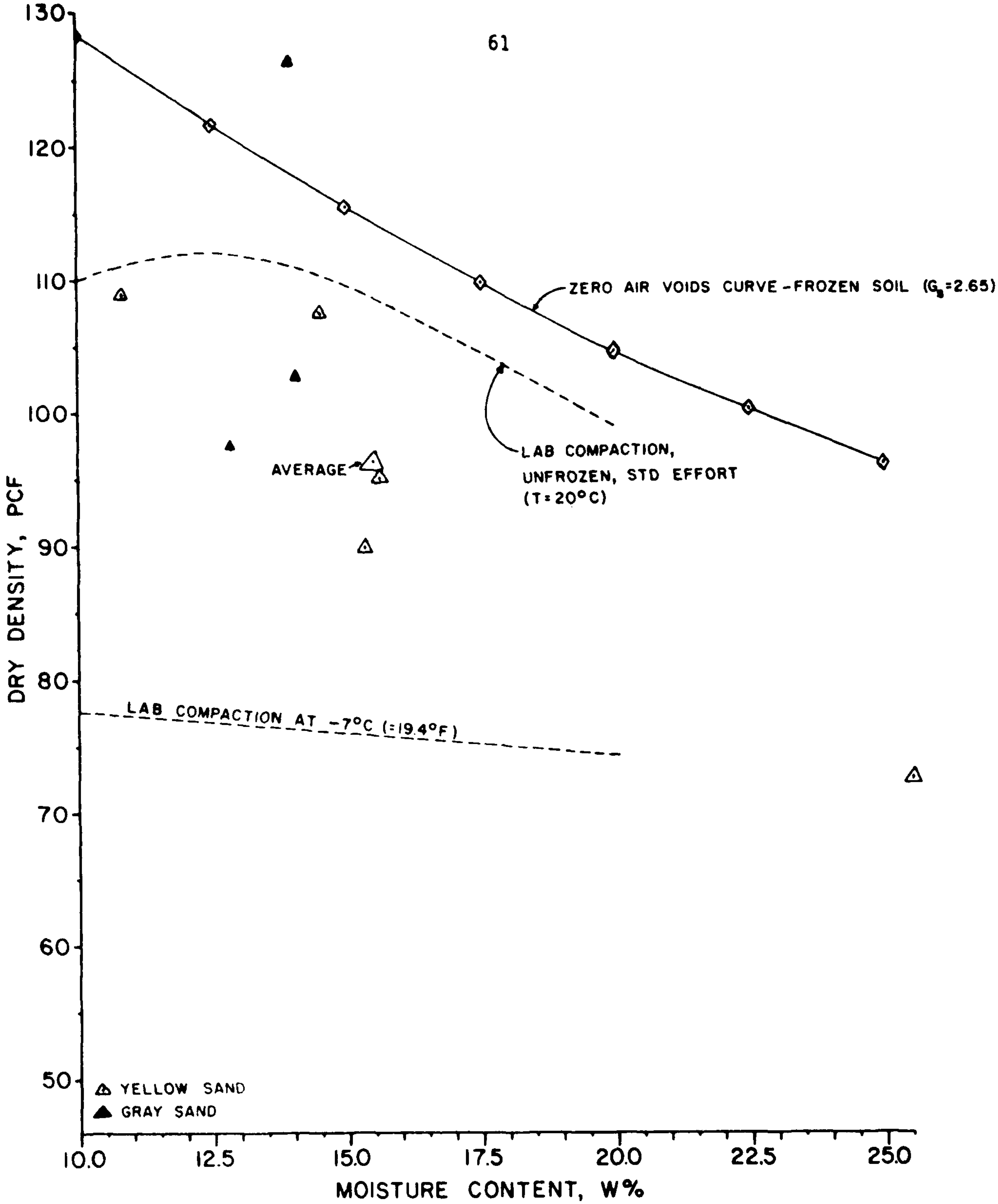

FIGURE VI-1B MOISTURE DENSITY RELATIONSHIPS - 4 PASSES OF ROLLER COMPARED TO LABORATORY COMPACTION

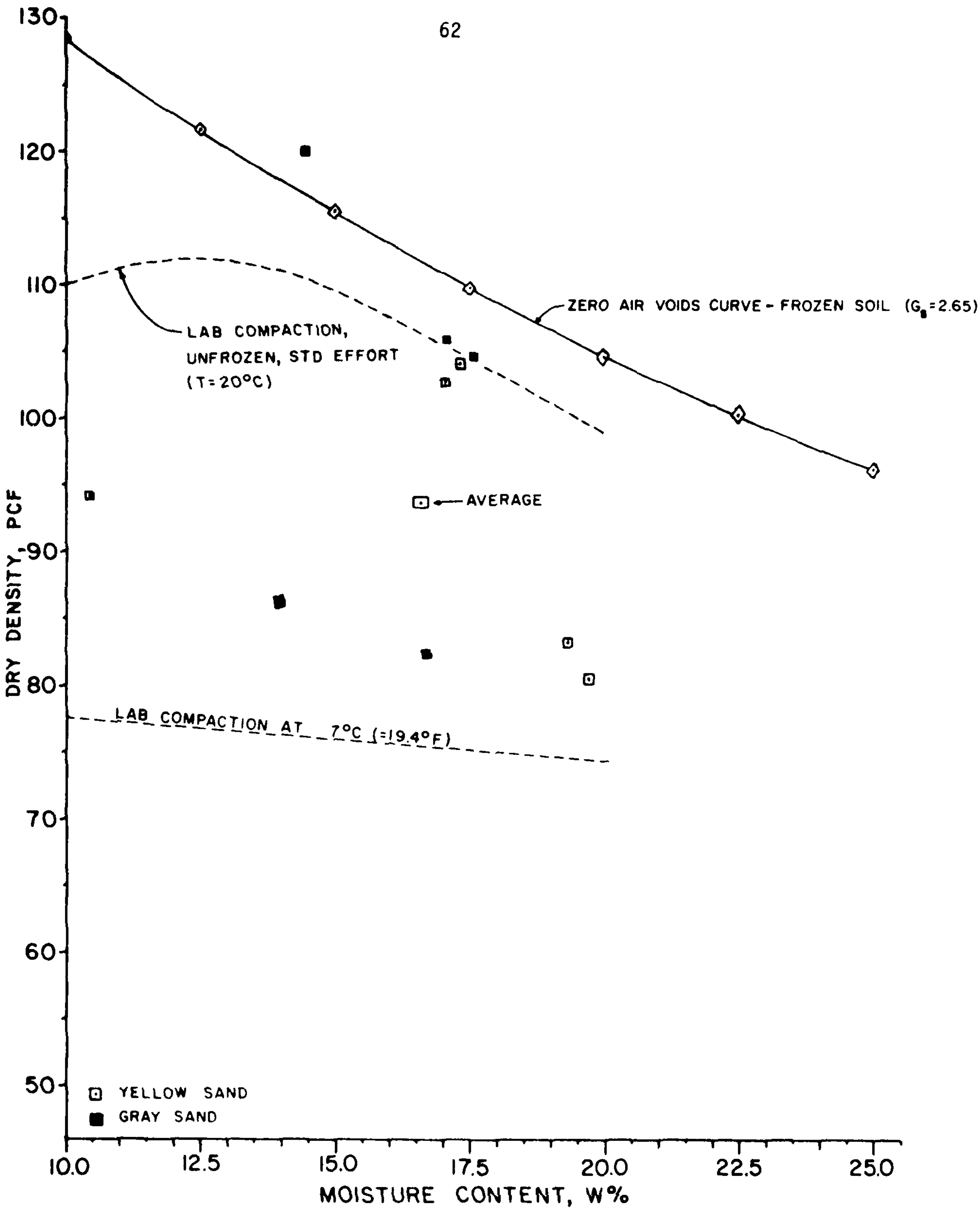

FIGURE VI-1c MOISTURE DENSITY RELATIONSHIPS - 10 PASSES OF ROLLER COMPARED TO LABORATORY COMPACTION

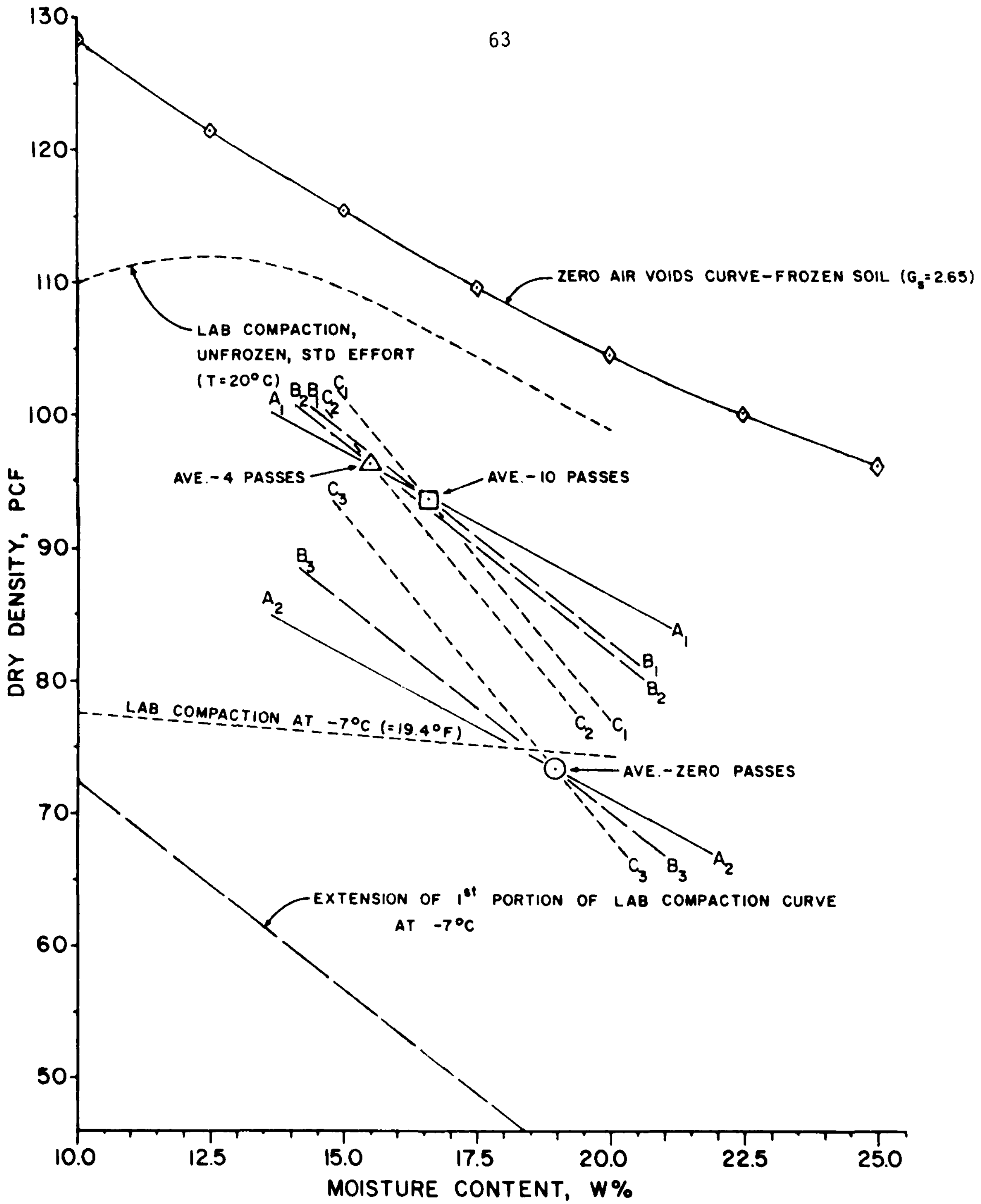

FIGURE VI-1b POSSIBLE INTERPRETATIONS OF FIELD COMPACTION DATA - INCREASE IN DENSITY WITH GREATER COMPACTIVE EFFORT

the frozen compaction curve, which actually applies only to the moisture range from zero to 8.2 percent. The two curves dealing with frozen compaction were adapted from Figure V-5, page 37 of this report, and the compaction curve for the unfrozen soil was adapted from the lab curve for the yellow sand.

Also shown on Figure VI-1d are three groups of curves, any group possibly representing the dependency of compacted dry density on both water content and compactive effort.

Curve A_1 is drawn essentially parallel to the zero air-voids curve, and passes through the points for both 4 passes and 10 passes. This would be equivalent to assuming that 6 additional passes would contribute no additional density. Although this does not seem reasonable, the slope of the plotted line suggests one possible limiting condition. Curve A_2 has been drawn through the point for zero passes, parallel to A_1, for comparison with other interpretations.

Because this interpretation does not indicate any advantage of 10 passes over 4, another set of lines was drawn parallel to the initial portion of the frozen compaction curve. Thus B_1 would represent the density-water content relationship for 10 passes, B_2 for 4 passes, and B_3 for zero passes. This would indicate a modest increase in density from 4 to 10 passes, but the selection of the slope of the line is rather arbitrary.

The C-group of lines was drawn such that the spacing between C_1 and C_2 is about half that between C_2 and C_3. This assumes that 6 additional passes would produce 50 percent as much increase in density as the first four passes would. Again, this is arbitrary, but it does provide some framework for evaluating the limited data.

There is some support, however, for the belief that additional compactive effort can produce significant increases in density. Laboratory compaction curves for frozen soil (-7°C) from the previous study by Haas, Alkire and Kaderabek (11)

show that at moisture contents in the range of 16 to 18 percent, the modified procedure can produce dry densities several pcf greater than the standard compactive effort can. This is shown in Figure VI-2a, which is adapted from Figure IV-8, page 38 and Figure IV-9, page 41 of Reference (11). These curves apply to a different soil from those used in the test embankment, and are from a lab test rather than a field test, thus cannot be directly applied to the present case. However, neither should this evidence be ignored.

In summary, although there are too many variables and not enough test data to draw firm conclusions about the effectiveness of field compaction of frozen soils, it is believed that some trends may be discernible. There are several reasons for the apparent effectiveness of the field compaction at low temperature in comparison to the laboratory test at low temperature.

1) Laboratory tests are conducted under controlled conditions with little fluctuation in temperature. The field tests, on the other hand, were subject to daily variations in temperature. Generally, the temperature in the morning was in the 20's (^{o}F) with temperatures at or near freezing being common in the afternoons. Temperatures near the surface of the soil may have been above freezing during some of the field compaction resulting in higher dry densities than would be obtained at a constant temperature below freezing as was the case with the laboratory tests.

2) The effect of gradation may have contributed to the higher dry densities obtained from field compaction. It has been shown in Figures V-8 and V-9 that the soil placed in the embankment had a fair distribution of particle sizes in comparison to the single particle used in the laboratory tests. Thus, according to the minimum voids concept the dry density for the field tests should be higher.

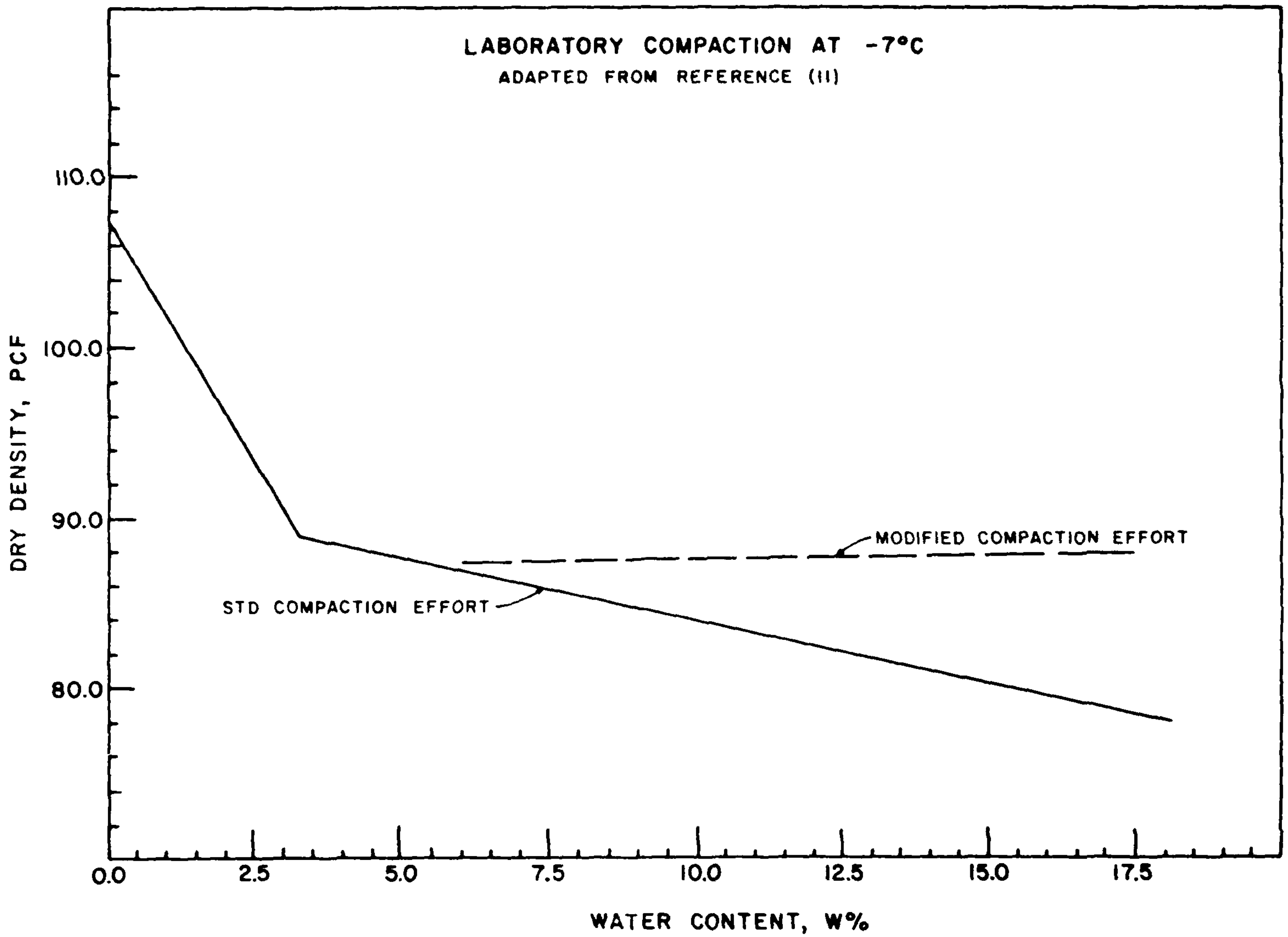

FIGURE VI-2A COMPARISON OF DENSITIES OBTAINED BY TWO COMPACTIVE EFFORTS APPLIED TO FROZEN SOIL

6.3 Foundation Settlement

A major consideration in prohibiting winter earthwork is the expectation that large settlements will occur upon thawing. Observations from the test embankment tend to confirm this belief. It was noted that total settlement, as observed from the surface markers, approached 0.2 feet at Station 8+70; however, approximately 25% of the total settlement was due to deep seated movements within the foundation soil.

The classification and extent of the foundation soil is not known because only surface samples and shallow borings were taken during the exploration stage. Therefore no attempt was made to predict the settlement in the subgrade material. Actual settlements that occurred were determined from settlement plates located at the surface of the foundation soil. Level readings taken on these settlement plates gave displacements within the foundation soil of 0.00 feet to 0.11 foot as was shown in Table V-3. The apparent settlement was non-uniform and reflects: 1) generally a higher level of fill in the Northwest portion of the embankment, and 2) possible weaker foundation conditions in the Northwest area due to the swampy area north of Station 8+40. It appears that a reasonable estimate of the average value of settlement for the foundation soil is within the range of 0.05 foot.

6.4 Embankment Settlement

The major part of the total measured settlement was due to the settlement in the embankment itself. Calculation of embankment settlements presents major difficulty if soil parameters are not known accurately. However, once the soil parameters are available, methods of analysis based on elastic theory can be used to evaluate displacements that take place within an embankment (7).

6.4.1 Modulus of Elasticity

Utilization of elastic methods for determination of embankment displacements is highly dependent on the value of dry density and modulus of elasticity and to a lesser degree on Poisson's ratio. Direct determination of the modulus in the field would have been the optimum condition. However, this was impractical since the fill displacements were closely related to the depth of thaw and it would have been impossible to determine the thawed properties of the soil until late spring. At this time the initial placed conditions would no longer exist in the fill due to the consolidation of the fill. Therefore, Young's Modulus was calculated using the constrained modulus obtained from consolidation tests conducted on unfrozen embankment soils and assumed values of Poisson's ratio.

Using one-dimensional consolidation test results, the constrained modulus can be calculated as:

$$E_c = \frac{\Delta\sigma}{\Delta\varepsilon_v} \qquad (1)$$

where $\Delta\sigma$ equals the increment of applied stress and $\Delta\varepsilon_v$ equals the increment of volumetric strain. For a one-dimensional consolidation test the volumetric strain is equal to the change in void ratio (e) divided by $(1+e_o)$ where e_o is the initial void ratio for the increment. In addition, the coefficient of compressibility $(a_v = \frac{\Delta e}{\Delta\sigma})$ can be used to reduce Equation 1 to form:

$$E_c = \frac{1 + e_o}{a_v} \qquad (2)$$

The stress levels in the test embankment are quite low and are conservatively estimated as being .235 tsf (4.7 ft. x 100 lbs./ft.^{3}). This value of stress was used as $\Delta\sigma_v$, along with e_o and a_v obtained from the laboratory tests, in Equation 2 to calculate the constrained modulus. The curve in Figure VI-3 is based on published analysis as well as the data, and of initial dry density on the constrained molulus. A value of 8.8 tsf was obtained as the average embankment constrained modulus of

elasticity from this figure using the average as-constructed density of the embankment soil (93 pcf).

Once the constrained modulus is known, Young's Modulus can be determined from the expression:

$$E = \frac{(1 + \mu)\ (1 - 2\mu)}{(1 - \mu)} E_c \qquad (3)$$

where μ = Poisson's ratio. Poisson's ratio is not, in general, a constant and varies during the loading process; however, Poisson's ratio can be estimated for various loadings and initial conditions. Lambe and Whitman (21) suggest that during the early parts of a triaxial compression test on a sandy soil, Poisson's ratio is low (.1 - .2) and increases as the particles are rearranged into a more compact configuration. A value of .3 was selected for use in Equation 3 to obtain Young's Modulus for calculation of embankment settlement.

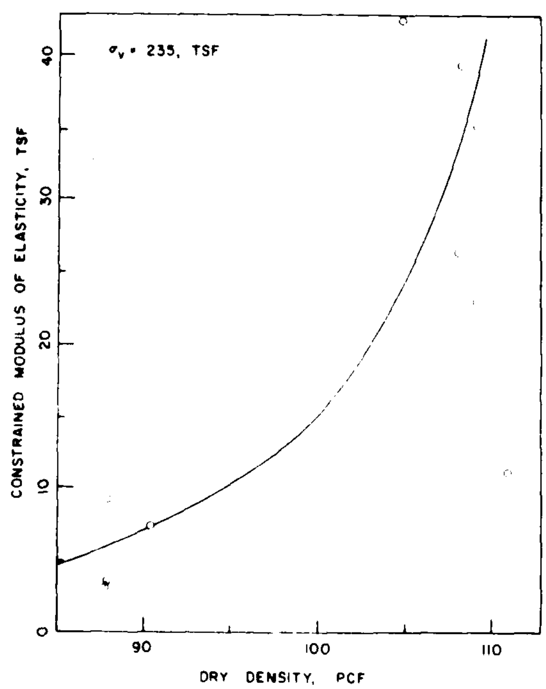

FIGURE VI-3 CONSTRAINED MODULUS OF ELASTICITY VERSUS DRY DENSITY OF THE EMBANKMENT SOILS.

6.4.2 Calculation of Settlement

In evaluation of displacements in an embankment it is necessary to differentiate between the classical definition of displacement referred to a fixed datum and displacements observed during incremental construction of the embankment. The former, called "single lift", is based on the assumption that the entire weight of the embankment is applied instantaneously and the displacement would be the elemental strains integrated over the full height of the embankment. Observed or "incremental" displacements, on the other hand, are due to accumulated strains occurring at a point as additional layers, or increments, of soil are added above the point of interest. Generally the incremental approach would more nearly model actual field construction practice of an embankment. However, an embankment constructed from frozen soil would very nearly simulate the single lift method of construction because frozen soil exhibits only limited compressibility during construction. This can be verified from Figures V-12 and V 13 which show little displacement until thawing commenced.

To evaluate displacements using the "single lift" concept, the embankment section at Station 8+70 is idealized as shown in Figure VI-4 and the settlements of the embankment (rigid foundation) calculated by an expression obtained from Poulos and Davis (31):

$$\rho\left(\frac{z}{H}, \frac{h}{H}\right) = I\left(\frac{z}{H}, \frac{h}{H}\right) \gamma \frac{H^2}{E} \tag{4}$$

where ρ = the single lift displacement at the centerline of the embankment, γ = unit weight of the embankment material, H = maximum height of embankment for a given base width and side slope, z = distance above datum (base of embankment) to the point of interest, h = distance above datum to top of embankment, E = modulus of elasticity and I = an influence factor, a

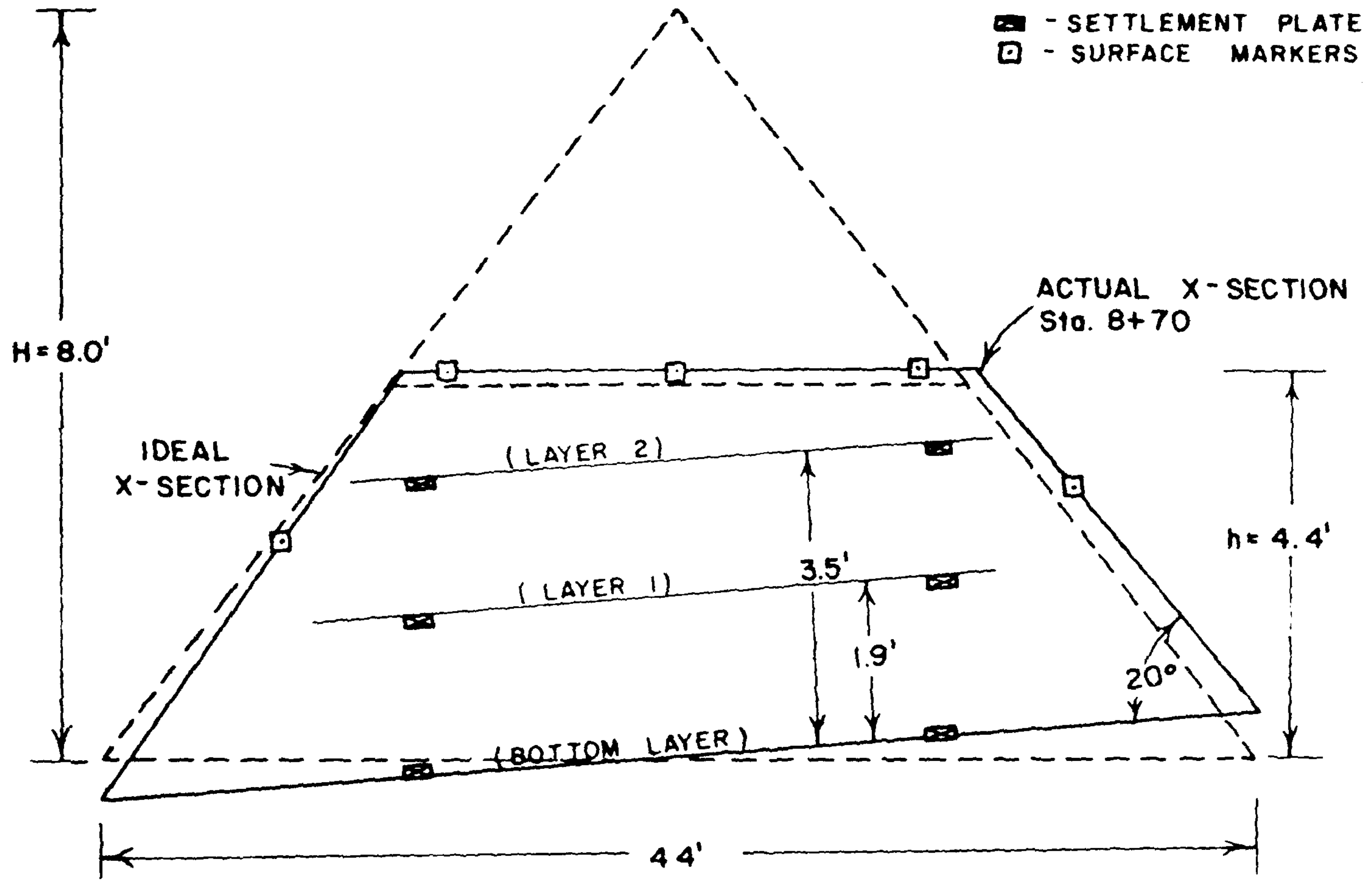

FIGURE VI-4 IDEAL AND ACTUAL CROSS-SECTIONS OF THE EMBANKMENT.

function of the embankment dimensions.

Equation 4 has been solved for displacements at three elevations along the embankment centerline, and the results are tabulated along with the measured displacements in Table VI-1. The average measured values shown in this table were obtained from the measured displacements of the settlement plates on the east and west edge of the embankment at the given station. If the observed displacements are corrected by deducting the settlement of the foundation soil, the difference between the calculated values and the average observed values is very small. The comparisons stated above assume the displacement at the edge and centerline at the top of the fill are the same. This is not correct in terms of rigorous mathematical analysis. However, the difference in calculated values for centerline and edge is small and for the small displacements encountered in this fill may be considered to be inconsequential.

The results tabulated in Table VI-1 indicate that an embankment using frozen soil, then allowed to thaw, behaves much like a "single lift" embankment constructed from unfrozen soil. Therefore, the key to limiting displacements within an embankment constructed from frozen soil is to obtain the highest possible unit weight (and highest modulus of elasticity) by adequate compaction of the soil.

6.4.3 FEASTS Computer Solution

Settlements in the embankment were also calculated with the aid of Massachusetts Institute of Technology's FEASTS finite element computer solution.

The actual cross-section on Station 8+70 was divided into 12 elements which resulted in 20 nodal points in the cross-section which were investigated for deformation.

TABLE VI-1 OBSERVED VERSUS CALCULATED EMBANKMENT SETTLEMENTS

STATION		Average Measured Displacement, feet				Calculated Displacement, feet		
		Bottom	Top Layer 1	Top Layer 2	Surface	Top Layer 1	Top Layer 2	Surface
8+40	Total	0.06	0.12	0.13	0.16	0.03	0.05	0.06
	Cor.*	0.00	0.06	0.07	0.10			
8+70	Total	0.07	0.08	0.13	0.15	0.03	0.05	0.06
	Cor.*	0.00	0.01	0.06	0.08			
9+00	Total	0.02	0.09	0.09	0.10	0.03	0.05	0.06
	Cor.*	0.00	0.07	0.07	0.08			

Cor.* = Corrected (Total Settlement - Foundation Settlement)

The material properties of the fill were varied to determine the effect of dry density and Poisson's ratio on the settlement characteristics of the soil. The dry density parameter was incorporated into the program by obtaining an E_c for a given dry density from Figure VI-3 and calculating E. The results for two surface points are presented in Table VI-2. The height of the fill at Station 8+70 was 4.4 feet at the centerline and 4.5 feet at 8 feet west of the centerline. This accounts for the larger settlement at the latter location.

The computer solution resulted in an average settlement of 0.05 foot for the range of Poisson's ratios selected, and for a soil at a dry density of 93 pcf. The settlement estimated by the elastic method was also 0.05 foot at 93 pcf. However, the observed settlement was 0.08 foot at Station 8+70. Therefore the computer results and the calculated settlements were slightly smaller than the observed settlements.

It should be noted that the value of 93 pcf used as the average initial embankment dry density was the average of highly scattered data. It appears that the value of 93 pcf was an over-estimation of the actual in situ dry density. From Table VI-2 it can be noted that inserting a dry density of approximately 88 pcf in the computer solution will result in a settlement of 0.08 foot, equaling the observed settlement. This is also true of the elastic solution.

Therefore, it is of great importance that the parameters of the embankment soil be estimated correctly to insure an accurate estimate of the deformation in a thawing embankment. Of these parameters, the dry density has the most effect on the calculated settlement and from Table [illegible] it is evident that Poisson's ratio has no appreciable effect on the [illegible] of settlement.

Table VI-2. FEASTS COMPUTER SOLUTION RESULTS (Station 8+70)

E_c, psf	γ_d, pcf	Settlement @ CL, inches		
		$\mu = .2$	$\mu = .3$	$\mu = .4$
9,200	85	.095	.104	.106
14,000	90	.062	.068	.070
17,600	93	.050	.054	.055
20,400	95	.043	.047	.048
30,000	100	.029	.032	.032

E_c, psf	γ_d, pcf	Settlement @ 8 ft. West of CL, inches		
		$\mu = .2$	$\mu = .3$	$\mu = .4$
9,200	85	.102	.113	.119
14,000	90	.067	.074	.078
17,600	93	.053	.059	.062
20,400	95	.046	.051	.054
30,000	100	.031	.035	.037

$$E = E_c \left[\frac{(1+\mu)\ (1-2\mu)}{1-\mu} \right]$$

VII. SUMMARY OF RESULTS

The project described in this report was an investigation designed to identify basic problems associated with winter earthwork. The field work included excavation of frozen soil and placing and compacting the excavated soil in an embankment. Suitable instrumentation and tests were conducted to aid in assessing the effectiveness of the excavation and compaction techniques. A substantial effort was also made to determine embankment displacements as the soil thawed. The results obtained during each of the phases of field work are described in more detail below.

7.1 Ripping (Excavation of Frozen Soil)

Excavation of the soils used for the embankment was done using a D7 tractor with a single blade-mounted ripping tooth. Of primary interest during this phase of the construction activities were the determination of the efficiency of the ripping operation and the size of the soil chunks produced by the ripping. Some of the results obtained during this phase are based on observation made in the field and others are the result of tests conducted during the ripping operation. The primary observations obtained from the ripping operation are as follows:

1. Ripping frozen soil is difficult and requires heavy equipment. In many cases the D7 used during this project could not penetrate the frozen crust or had inadequate power to rip the soil once the ripping tooth had penetrated the soil.

2. Because of the limited ability of the D7, it was impossible to establish any predetermined pattern for the ripping operation. Even very close spacing of individual passes with the ripping tooth did not insure success. The only method that worked effectively was to find a weak spot in the frozen crust (areas of low water content, protected by a cover of snow, or granular soil) and work out from this point in the path of least resistance.
3. Ripping of frozen soil produced a large variation in chunk sizes. Chunks up to 6 ft. in diameter were not uncommon. Most of the larger chunks were platy with the smallest dimension approximately equal to the depth of the frozen crust. Larger chunks could be degraded by crushing under the weight of the dozer.
4. Gradation analysis of the soil used in the embankment indicated a wide distribution of sizes, ranging from chunks of 2 ft. in diameter to individual soil particles less than 1/4 in. in diameter.

7.2 Field Compaction

Field compaction of the embankment was completed using a sheepsfoot roller. Densities of the embankment material were taken before rolling commenced and after 4 and 10 passes with the sheepsfoot roller. Analysis of the field density results indicates:

1. Field densities after 10 passes with the roller were comparatively high. The average density for the entire embankment was 93 pcf at a moisture content of 16 percent.

This is approximately 80 percent of maximum dry density obtained in the laboratory for the unfrozen soil and 125% of the dry density obtained in the laboratory for compaction of the soil when frozen.

2. Effectiveness of the field compaction was highly dependent on the moisture content of the soil. Field densities near the laboratory maximum were obtained as the moisture content approached the laboratory optimum moisture content.
3. Wide variations in moisture content can be expected when compacting frozen soils in the field. Ambient conditions make it impossible to use any of the normal "summer time" methods for control of moisture (adding water with sprinklers or aeration by disking).
4. Frozen soils compacted in the field were responsive to increasing compactive effort. Pre-rolling densities were approximately 78 pcf; these increased to 93 pcf after 10 passes with the roller.

7.3 Post-Thaw Settlement

Settlement of the soil as thawing took place was monitored using settlement plates placed throughout the embankment. Correlation of thawing and displacement was made using frost tubes and ground temperature data obtained from thermistors. The main observations concerning post-thaw displacements within the embankment include:

1. Displacements will be comparatively large. For the test embankment maximum observed displacements were in the order of 0.1 ft. for the 4 plus ft. of fill. (Vertical strain = 2.5%)

2. The magnitude of embankment displacement is closely related to the compacted density of the frozen soil.. Embankment displacement can be predicted using theoretical methods based on elasticity as was done in this project.
3. The development of displacements within the embankment was closely related to thawing of the embankment soil. No displacements were observed until after the ground temperature had risen above $32^{o}F$.

VIII. CONSTRUCTION RECOMMENDATIONS

The experience gained through completion of this project suggests several recommendations that might be useful in completion of the various phases of winter earthwork.

8.1 Excavation of Frozen Soils

1. Frozen soils are very tough and require heavy equipment if they are to be successfully ripped. The D7 used in this project was adequate; however, a more efficient job could have been done with a more powerful dozer. Along with a heavier dozer a better ripper would increase production. The single tooth, blade mounted ripper used during this project is the minimum piece of equipment that should be used in ripping frozen soil. Rear mounted rippers (Caterpillar No. 7 or equivalent) would probably have been much more efficient.
2. Ripping operations should be started in an area where the depth of frozen soil is the smallest. Even in areas exposed to similar environmental conditions, there will be areas of comparatively weak frozen soils due to localized differences in snow cover, moisture content, or soil type. These areas can be detected during the initial site investigation and should be fully exploited.
3. Ripping will produce chunks of frozen soil that will vary in size from very large (6 ft.) to the size of individual soil particles. The large chunks may be degraded by the weight of the construction equipment and the borrow area should be worked as much as possible. Very large chunks, unsuitable for construction, may be cast to the edge of the borrow areas and may be used as supplemental fill after

they have thawed.

8.2 Compaction of Frozen Soil

1. Plan compaction activities to coincide with the highest daily temperature (usually during the afternoon). During sunny days the temperature near the ground may be well above the average daily temperature and will make compaction more efficient.
2. Frozen soils can be compacted although it will be unlikely that densities will be as high as obtained by similar compaction during "summer time". Initial attempts to compact frozen soils should be closely monitored. A graph of frozen dry density versus number of passes can be developed and used to select the optimum amount of compactive effort that should be applied to the soil.
3. Recompaction of the fill after thawing may be required to obtain acceptable densities. There is an indication that consolidation during settlement may substantially increase the as-compacted density and the amount of required recompaction may be minimal.
4. Good control of compaction will be hard to achieve during cold weather since wide variations in moisture content and environmental conditions will cause different dry densities for equal compactive efforts, thus quality controls may have to be relaxed. In addition, determination of in-place unit weights will be difficult to obtain by any method that requires a hole to be dug in the frozen soil (sand cone, balloon method, oil displacement, etc.) Nuclear density meters may be a good solution to this problem. However, this requires field verification.

8.3 Settlement Control

1. The magnitude of post-thaw settlements will be closely related to the

dry density obtained during the compaction of the frozen soil. Preliminary estimates of settlements can be made using elastic theory and soil parameters determined from the as-compacted density of the frozen soils.

2. The rate of settlement is dependent on the rate of thawing of the frozen material with a continuing settlement until the embankment is completely thawed. For the low embankment (4 plus ft.) constructed as part of this project, it took approximately two months (from late March to late May, 1975) to completely thaw the material placed during the winter. Since it would be inadvisable to use the embankment as a foundation until the thaw settlement was completed, this may cause serious delays in construction (assuming a pavement was to be placed on the fill). This problem seems to be unavoidable and suggests that embankments constructed during the winter would be most appropriate in stage construction where earthwork is completed well in advance of the structural part of the project.

APPENDIX A

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APPENDIX B

Field Density Data

BORROW AREA FIELD DENSITIES

Location	Dry Denisty γd, pcf	Moisture Content w%
Sta. 8+09 CL	96.2	10.9
Sta. 8+51 25 ft. E CL	109.0	12.5
Sta. 8+51 45 ft. E CL	97.1	15.1
Sta. 8+65 55 ft. W CL	115.2	3.9
Sta. 8+82 50 ft. W CL	119.5	17.4
Sta. 8+97 64 ft. W CL	98.3	13.9
Sta. 9+00 50 ft. E CL	88.1	12.6
Sta. 9+15 40 ft. E CL	140.5	8.2
Sta. 9+25 50 ft. E CL	95.4	12.6
Sta. 9+29 CL	138.8	11.8
Sta. 9+29 CL	122.9	9.4

Samples taken over depth range 0.5 to 2 feet

FILL AREA FIELD DENSITIES

Location	Dry Density γd, pcf	Moisture Content w%
Sta. 8+50 28 ft. E CL	115.5	9.2
Sta. 8+82 28 ft. E CL	99.0	21.5
Sta. 8+82 30 ft. W CL	180.7 (Bad test-not used)	7.3
Sta. 9+25 30 ft. W CL	124.7	13.1
Sta. 9+25 30 ft. E CL	99.1	17.1
Sta. 9+50 30 ft. E CL	88.2	21.4
Sta. 9+50 30 ft. W CL	96.6	18.7
Sta. 9+85 CL	101.5	19.9

Samples taken from surface to depth of 1.0 foot

AD-A086 877 COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER NH F/G 13/13
THE CONSTRUCTION OF AN EMBANKMENT WITH FROZEN SOIL.(U)
MAY 80 J J BOTZ, W M HAAS
UNCLASSIFIED CRREL-SR-80-21 NL

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APPENDIX C

Total Observed Settlement of Surface Markers

	Location	March 26	28	31	April 2	4	9	11	14
Sta. 9+00	17 ft. W CL	.01	.02	.00	.01	.00	.02	.02	.02
	12 ft. W CL	.00	.01	-.01	.00	-.01	.00	.00	.00
	CL	-.02	-.01	-.02	-.02	-.01	-.01	-.02	-.01
	6 ft. E CL	-.01	-.01	-.01	.00	.02	.02	.02	.02
	12 ft. E CL	.00	.00	.01	.00	.00	.02	.02	.02
Sta. 8+80	16 ft. W CL	.01	.01	.00	.01	.00	.02	.01	.02
	10 ft. W CL	.00	.01	.00	.01	.00	.02	.01	.02
	CL	.00	.01	.00	.00	.00	.00	.00	.00
	7 ft. E CL	.00	.01	.00	.00	.00	.01	.00	.00
	12 ft. E CL	.01	.01	.01	.01	.01	.01	.01	.01
Sta. 8+70	15 ft. W CL	.02	.01	.02	.02	.02	.03	.03	.04
	8 ft. W CL	.01	.01	.00	.01	.01	.02	.01	.02
	CL	.01	.01	.01	.01	.00	.01	.01	.01
	7 ft. E CL	.01	.01	.01	.01	.00	.01	.00	.01
	14 ft. E CL	-.01	.01	.00	-.01	-.01	.01	.02	.00
Sta. 8+60	14 ft. W CL	.00	.00	.00	.01	.01	.03	.02	.03
	7 ft. W CL	-.01	.00	-.01	-.01	.00	.01	.01	.01
	CL	-.01	-.02	-.02	-.02	-.01	.00	-.01	.00
	10 ft. E CL	-.02	-.02	-.01	-.02	-.01	.00	.00	.00
	15 ft. E CL	.00	-.01	.01	-.01	-.01	.01	.00	.01
Sta. 8+40	12 ft. W CL	-.01	.00	.00	-.01	-.01	.01	.01	.01
	6 ft. W CL	.00	.00	.01	.00	.00	.02	.01	.02
	CL	-.01	-.01	-.01	-.01	-.02	.00	.00	.00
	12 ft. E CL	-.01	-.01	.00	-.01	-.02	.00	.00	.00
	17 ft. E CL	-.01	-.02	-.01	-.01	-.02	.01	.00	.01

(Appendix C continued)

	Location	April 16	18	21	25	28	May 2	5	9
Sta. 9+00	17 ft. W CL	.03	.03	.03	.04	.03	.02	.03	.04
	12 ft. W CL	.01	.01	.02	.04	.03	.04	.06	.09
	CL	.00	.00	.01	.02	.01	.01	.03	.05
	6 ft. E CL	.02	.02	.03	.05	.05	.05	.06	.07
	12 ft. E CL	.03	.03	.03	.05	.03	.03	.03	.04
Sta. 8+80	16 ft. W CL	.02	.02	.02	.02	.01	.01	.02	.02
	10 ft. W CL	.02	.03	.05	.07	.07	.08	.09	.09
	CL	.00	.00	.02	.04	.03	.04	.05	.06
	7 ft. E CL	.01	.00	.01	.05	.04	.04	.05	.06
	12 ft. E CL	.01	.00	.01	.05	.04	.04	.05	.06
Sta. 8+70	15 ft. W CL	.04	.05	.05	.06	.05	.05	.06	.06
	8 ft. W CL	.02	.04	.05	.06	.06	.06	.07	.08
	CL	.01	.03	.05	.05	.05	.05	.06	.07
	7 ft. E CL	.01	.01	.04	.05	.05	.05	.06	.08
	14 ft. E CL	.02	.03	.05	.05	.05	.05	.06	.07
Sta. 8+60	14 ft. W CL	.03	.03	.04	.05	.04	.04	.05	.05
	7 ft. W CL	.04	.05	.06	.07	.07	.08	.09	.11
	CL	.00	.01	.02	.03	.02	.03	.04	.05
	10 ft. E CL	.00	.00	.01	.02	.02	.03	.04	.07
	15 ft. E CL	.02	.04	.06	.07	.06	.06	.07	.08
Sta. 8+40	12 ft. W CL	.02	.02	.03	.03	.03	.03	.04	.04
	6 ft. W CL	.02	.02	.04	.05	.05	.06	.07	.08
	CL	.00	.01	.02	.03	.03	.04	.05	.06
	12 ft. E CL	.00	.00	.01	.02	.02	.02	.04	.06
	17 ft. E CL	.02	.01	.03	.03	.03	.04	.04	.06

(Appendix C continued)

	Location	May 12	16	19	23	June 3	18	July 10
Sta. 9+00	17 ft. W CL	.03	.03	.03	.04	.03	.03	.09
	12 ft. W CL	.10	.10	.10	.11	.11	.11	.12
	CL	.07	.07	.08	.10	.10	.10	.10
	6 ft. E CL	.08	.08	.08	.09	.09	.11	.10
	12 ft. E CL	.04	.05	.03	.04	.02	.03	.03
Sta. 8+80	16 ft. W CL	.02	.02	.02	.03	.02	.03	.02
	10 ft. W CL	.12	.13	.14	.14	.18	.18	.18
	CL	.07	.08	.08	.11	.12	.13	.13
	7 ft. E CL	.06	.07	.07	.09	.10	.10	.10
	12 ft. E CL	.08	.08	.08	.09	.08	.09	.08
Sta. 8+70	15 ft. W CL	.07	.06	.06	.07	.06	.08	.07
	8 ft. W CL	.09	.09	.11	.13	.14	.14	.14
	CL	.08	.08	.09	.12	.14	.13	.15
	7 ft. E CL	.08	.08	.09	.12	.12	.13	.13
	14 ft. E CL	.06	.06	.06	.05	.06	.06	.06
Sta. 8+60	14 ft. W CL	.07	.05	.05	.06	.06	.05	.06
	7 ft. W CL	.12	.14	.15	.18	.18	.19	.19
	CL	.06	.07	.08	.09	.12	.12	.12
	10 ft. E CL	.08	.09	.09	.11	.10	.11	.11
	15 ft. E CL	.08	.07	.06	.08	.08	.08	.08
Sta. 8+40	12 ft. W CL	.04	.04	.04	.04	.04	.04	.04
	6 ft. W CL	.09	.10	.12	.15	.17	.19	.18
	CL	.06	.07	.09	.12	.16	.16	.21
	12 ft. E CL	.07	.07	.08	.09	.08	.09	.08
	17 ft. E CL	.05	.04	.04	.05	.05	.05	.06

APPENDIX D

Total Observed Settlement of Settlement Plates

	Location	March 26	28	31	April 2	4	7	9	11	14	16	18	21	25	28
Sta. 9+00	10 ft. W CL	(Not located until July 15)													
	9 ft. W CL	.00	.00	-.01	.00	.00	.01	.00	.00	.01	.00	.00	.01	.02	.01
	8 ft. W CL	-.01	-.01	-.02	.00	-.01	.01	.00	.00	.02	.01	.01	.02	.04	.04
	8 ft. E CL	.00	.00	-.01	.00	.00	.01	.01	.00	.01	.01	.01	.03	.05	.04
	9 ft. E CL	.01	.01	.00	.01	.00	.00	.01	.00	.01	.00	.01	.02	.04	.04
	10 ft. E CL	.01	.01	.00	.01	.00	.00	.01	.00	.01	.00	.01	.02	.04	.04
Sta. 8+70	10 ft. W CL	(Not located until July 15)													
	9 ft. W CL	.02	.03	.03	.03	.03	.05	.04	.03	.05	.04	.04	.05	.05	.05
	8 ft. W CL	.01	.01	.01	.01	.01	.02	.01	.01	.03	.02	.03	.04	.05	.05
	8 ft. E CL	.01	.01	.00	.01	.00	.02	.01	.00	.02	.01	.01	.03	.06	.05
	9 ft. E CL	.01	.01	.01	.01	.01	.02	.02	.00	.02	.01	.01	.01	.02	.01
	10 ft. E CL	(Not located until July 15)													
Sta. 8+40	10 ft. W CL	.00	.00	.01	.00	.00	.01	.01	.01	.02	.01	.01	.02	.02	.02
	9 ft. W CL	.03	.03	.04	.03	.04	.04	.04	.03	.05	.04	.04	.04	.05	.05
	8 ft. W CL	.00	-.01	.00	-.01	.00	.01	.01	.01	.02	.02	.02	.03	.04	.04
	8 ft. E CL	-.01	-.01	-.01	-.01	-.01	.00	.00	.00	.01	.00	.00	.01	.02	.02
	9 ft. E CL	.02	.02	.02	.02	.01	.03	.02	.02	.03	.02	.02	.02	.03	.03
	10 ft. E CL	(Not located until July 15)													

(Appendix D continued)

Station	Location	May 2	5	9	12	16	19	23	June 3	18	July 10	15	Relative Height
Sta. 9+00	10 ft. W CL											.04	Foundation
	9 ft. W CL	.02	.04	.06	.08	.09	.09	.10	.10	.11	.10		1/3 H
	8 ft. W CL	.03	.05	.08	.09	.10	.10	.12	.11	.12	.11		2/3 H
	8 ft. E CL	.05	.06	.07	.07	.07	.07	.08	.08	.08	.07		2/3 H
	9 ft. E CL	.04	.05	.06	.07	.06	.06	.06	.07	.08	.07		1/3 H
	10 ft. E CL	-.01	.00	.00	.00	.00	.00	.00	.01	.01	.00	.00	Foundation
Sta. 8+70	10 ft. W CL											.11	Foundation
	9 ft. W CL	.05	.05	.07	.07	.08	.08	.10	.11	.09	.09		1/3 H
	8 ft. W CL	.04	.07	.07	.07	.08	.08	.12	.12	.12	.13		2/3 H
	8 ft. E CL	.06	.07	.09	.10	.10	.11	.13	.13	.14	.13		2/3 H
	9 ft. E CL	.01	.02	.03	.04	.05	.05	.07	.07	.07	.07		1/3 H
	10 ft. E CL											.03	Foundation
Sta. 8+40	10 ft. W CL	.02	.03	.03	.03	.03	.03	.03	.03	.03	.04	.04	Foundation
	9 ft. W CL	.05	.06	.08	.08	.09	.09	.11	.11	.10	.12		1/3 H
	8 ft. W CL	.04	.06	.07	.08	.08	.09	.12	.13	.13	.13		2/3 H
	8 ft. E CL	.03	.04	.06	.06	.08	.09	.12	.12	.12	.12		2/3 H
	9 ft. E CL	.02	.05	.06	.07	.09	.10	.11	.12	.12	.12		1/3 H
	10 ft. E CL											.08	Foundation

APPENDIX E

Temperature, ^{o}F, of West Set of Thermistors (Sta. 8+70)

Depth of Thermistor feet	March 21	26	28	31	April 2	4	7	9	11	14	16	18
0.5	32.1	26.8	28.5	27.7	28.9	29.1	30.9	31.2	32.4	39.6	41.6	35.7
1.0	31.9	28.4	28.2	27.5	27.0	28.1	28.9	30.3	31.0	31.7	32.6	32.6
1.5	31.7	30.0	28.3	27.4	28.6	29.1	30.0	30.8	31.0	31.1	31.3	31.4
2.0	31.8	31.1	30.2	29.6	29.0	29.4	30.6	30.3	30.7	31.0	30.9	31.1
2.5	31.7	31.0	31.0	30.6	30.1	30.3	30.0	30.5	30.9	30.6	30.9	31.0
3.0	31.7	31.4	31.4	31.2	30.8	31.0	30.3	30.6	31.2	31.0	31.0	31.0
3.5	31.8	30.5	31.4	31.5	31.3	31.0	30.5	31.1	31.0	31.2	31.0	31.1
4.0	31.9	31.6	31.6	31.9	31.7	31.7	31.3	31.4	31.7	31.6	31.3	31.5
4.5	31.9	31.7	31.8	32.0	32.0	31.0	31.4	31.6	31.6	31.8	31.4	31.6
5.0	32.1	32.1	32.3	31.7	32.5	32.6	32.0	32.2	32.5	32.3	32.0	32.1
5.5	32.7	32.6	32.7	33.0	32.8	33.0	32.6	31.9	33.0	32.9	32.5	32.8
6.0	33.3	32.7	32.7	34.0	33.5	33.5	33.0	33.2	33.4	33.4	33.0	33.2
6.5	34.0	33.4	34.9	34.4	33.9	34.2	33.7	33.9	33.8	34.0	33.6	33.6
7.0	34.7	34.3	34.6	35.3	34.7	34.7	34.3	33.9	34.4	34.4	34.1	34.2
7.5	35.2	35.0	35.1	35.6	35.2	35.1	34.9	34.9	34.9	34.7	34.5	34.7
8.0	35.8	35.8	35.6	36.3	35.6	35.7	35.1	35.0	35.3	35.3	34.8	35.0

(Appendix E continued)

Depth of Thermistor	April			May							June	
feet	21	25	28	2	5	9	12	16	19	23	3	18
0.5	35.3	45.9	36.3	42.7	44.8	55.3	51.3	54.5	56.2	68.5	56.0	59.0
1.0	33.4	36.7	33.8	36.4	38.0	44.7	44.2	42.5	50.4	58.5	53.5	54.0
1.5	31.5	32.4	31.7	33.3	34.4	37.5	41.1	38.5	46.7	52.0	52.0	53.5
2.0	31.4	31.5	30.7	31.5	32.3	34.8	36.7	36.5	41.3	46.5	50.5	52.5
2.5	31.4	31.3	30.4	30.5	30.7	32.1	34.1	34.5	37.4	41.5	46.5	51.0
3.0	31.3	31.3	30.3	30.5	30.5	30.5	31.8	33.0	34.3	37.5	44.0	50.0
3.5	31.2	31.2	30.3	30.4	30.5	30.3	30.2	31.0	31.5	31.5	42.0	48.5
4.0	31.7	31.8	30.6	30.7	30.5	30.5	30.6	30.5	30.7	31.0	40.0	45.5
4.5	31.6	31.6	30.6	30.6	30.5	30.5	30.5	30.5	30.7	30.5	39.0	43.5
5.0	32.4	32.3	31.2	31.3	31.2	31.1	31.2	31.0	31.2	31.0	37.5	42.5
5.5	32.8	32.7	31.7	31.7	31.6	31.5	31.6	31.5	31.6	31.5	37.0	41.5
6.0	33.3	33.4	32.1	32.1	32.0	32.0	32.1	32.0	32.0	32.0	36.5	41.0
6.5	33.7	33.7	32.4	32.5	32.4	32.3	32.3	32.5	32.4	32.5	35.5	40.5
7.0	34.3	34.1	32.5	32.9	32.7	31.7	32.7	32.5	32.8	32.5	35.5	40.0
7.5	34.7	34.6	33.0	33.3	33.1	33.0	33.1	33.0	33.0	33.0	35.0	39.5
8.0	35.0	34.9	32.8	33.6	33.5	33.5	33.1	33.5	33.6	33.5	35.0	39.0

APPENDIX F

Temperature, ^{o}F, of Southwest Set of Thermistors (Sta. 8+40)

Depth of Thermistor feet	March 21	26	28	31	April 2	4	7	9	11	14	16	18
0.5	32.3	28.0	27.4	27.8	28.7	29.4	31.2	31.6	32.3	42.0	44.4	36.5
1.0	32.3	28.5	28.3	27.5	26.9	28.2	29.7	30.7	31.1	33.7	36.7	34.2
1.5	31.9	30.6	29.3	28.9	27.4	29.0	29.5	30.4	30.7	31.2	31.4	31.5
2.0	32.0	31.5	31.4	30.5	29.1	29.7	29.9	30.5	30.9	31.0	31.0	31.2
2.5	31.8	31.5	31.2	31.0	30.4	30.4	30.3	30.5	31.0	31.0	31.0	31.0
3.0	31.8	31.4	31.4	31.6	30.8	31.0	30.6	30.9	31.0	31.2	31.2	31.0
3.5	31.8	31.4	31.5	31.8	31.5	31.3	30.9	31.2	31.3	31.4	31.3	31.3
4.0	31.9	31.4	31.6	31.7	31.7	31.9	31.3	31.4	31.4	31.7	31.3	31.4
4.5	31.9	31.6	31.6	32.0	32.0	31.0	31.7	31.7	31.7	32.0	31.6	31.7
5.0	32.1	32.0	32.0	32.2	32.4	32.5	32.3	32.3	32.2	32.4	31.1	32.3
5.5	32.6	32.3	32.6	32.3	33.0	32.9	32.8	32.8	32.6	32.9	32.7	32.7
6.0	33.3	33.3	33.3	33.7	33.6	33.8	33.6	33.6	33.5	33.7	33.3	33.3
6.5	34.0	34.0	34.0	34.1	34.3	34.2	34.0	34.2	34.0	34.1	33.6	33.7
7.0	34.7	34.6	34.6	34.9	34.9	35.0	34.7	34.7	34.7	34.7	34.3	34.3
7.5	35.1	34.9	35.3	35.2	35.3	35.5	35.5	35.2	35.0	35.2	34.7	34.7
8.0	36.0	35.7	35.4	35.7	35.7	35.9	35.7	35.6	35.3	35.7	34.7	35.2

(Appendix F continued)

Depth of Thermistor	April			May							June	
feet	21	25	28	2	5	9	12	16	19	23	3	11
0.5	37.3	47.3	37.1	44.6	45.8	56.5	53.2	54.5	58.8	71.5	57.5	58.5
1.0	36.2	40.5	36.0	39.7	41.5	49.0	47.9	47.5	54.1	64.5	56.0	55.5
1.5	32.0	34.2	33.8	35.5	36.7	40.5	43.4	42.5	50.7	57.5	56.0	54.0
2.0	31.3	33.3	31.7	33.1	34.0	37.3	39.2	40.0	47.0	53.5	55.0	54.0
2.5	31.2	31.5	30.3	31.0	31.8	33.9	36.1	37.0	41.0	48.5	53.5	54.0
3.0	31.3	31.2	30.0	30.5	30.4	31.4	33.0	35.0	37.7	43.5	51.5	52.0
3.5	31.3	31.3	30.3	30.5	30.3	30.4	31.0	33.0	35.8	41.5	50.5	51.0
4.0	31.6	31.5	30.5	30.6	30.5	30.5	30.6	31.5	34.6	39.5	49.5	50.5
4.5	31.6	31.7	30.7	30.9	30.7	30.7	30.9	31.5	33.8	38.5	46.5	49.5
5.0	31.2	32.3	31.2	31.4	31.2	31.4	31.5	32.5	32.6	37.5	44.0	48.0
5.5	32.7	32.7	31.5	31.7	31.6	31.7	32.0	32.5	33.8	36.5	43.0	47.5
6.0	33.5	33.3	32.0	32.3	32.2	32.4	32.5	33.0	33.9	35.5	41.5	44.5
6.5	34.0	34.7	32.5	32.7	32.6	32.7	32.8	33.5	34.1	35.5	41.0	44.0
7.0	34.5	34.2	32.8	33.1	32.6	33.1	33.3	33.5	34.3	35.5	40.5	43.0
7.5	34.9	34.6	33.4	33.5	33.3	33.5	33.6	33.7	34.4	35.5	39.5	42.5
8.0	35.6	35.0	33.7	33.7	33.6	33.7	34.0	34.0	34.6	34.5	39.0	42.5

APPENDIX G

Chunk Size Data

Area I, Station 9+50 25 ft. East of CL		
Least Dimension inches	Weight Passing pounds	% Passing
18	248.4	100.0
12	171.8	60.5
6	131.2	46.1
2	68.6	24.1
3/4	19.1	6.7

Area II, Station 9+25 25 ft. East of CL		
Least Dimension inches	Weight Passing pounds	% Passing
18	224.2	100.0
12	224.2	100.0
6	171.1	76.3
2	119.8	53.4
3/4	37.9	16.9

Area III, Station 8+80 10 ft. East of CL		
Least Dimension inches	Weight Passing pounds	% Passing
18	212.8	100.0
12	165.4	77.7
6	110.4	51.9
2	54.8	25.8
3/4	18.6	8.7

(Appendix G continued)

Area IV, Station 8+25 CL

Least Dimension inches	Weight Passing pounds	% Passing
18	124.0	100.0
12	124.0	100.0
6	102.2	82.4
2	82.0	66.1
3/4	38.0	30.6

Area V, Station 8+25 25 ft. East of CL

Least Dimension inches	Weight Passing pounds	% Passing
18	234.7	100.0
12	158.7	67.6
6	102.4	43.6
2	48.3	20.6
3/4	18.1	7.7

Area VI, Stockpiled

Least Dimension inches	Weight Passing pounds	% Passing
18	203.6	100.0
12	128.0	62.9
6	92.9	45.6
2	55.7	27.4
3/4	22.4	11.0

(Appendix G continued)

Area VII, Stockpiled

Least Dimension inches	Weight Passing pounds	% Passing
18	137.1	100.0
12	79.0	57.6
6	69.1	50.4
2	47.8	34.8
3/4	18.3	13.3

Area VIII, Stockpiled

Least Dimension inches	Weight Passing pounds	% Passing
18	179.2	100.0
12	109.2	60.9
6	86.1	48.0
2	55.3	30.9
3/4	23.4	13.1

Area IX, Stockpiled

Least Dimension inches	Weight Passing pounds	% Passing
18	173.0	100.0
12	133.9	77.4
6	93.5	54.0
2	77.9	45.0
3/4	35.3	20.4

APPENDIX H

Consolidation Test Data

E_o = .496 — Dry Density = 110 pcf — Moisture Content = 15%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.000	0.3000	2.500	0.3174
0.156	0.3052	5.000	0.3226
0.313	0.3072	10.000	0.3289
0.625	0.3099	20.000	0.3367
1.250	0.3136		

E_o = .537 — Dry Density = 108 pcf — Moisture Content = 15%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.000	0.3000	2.500	0.3218
0.156	0.3074	5.000	0.3276
0.313	0.3100	10.000	0.3342
0.625	0.3131	20.000	0.3422
1.250	0.3174		

E_o = .827 — Dry Density = 91 pcf — Moisture Content = 17%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.000	0.3000	2.500	0.3914
0.156	0.3226	5.000	0.3997
0.313	0.3409	10.000	0.4066
0.625	0.3557	20.000	0.4447
1.250	0.3752		

E_o = .556 — Dry Density = [illegible]06 pcf — Moisture Content = 17%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.000	0.3000	1.250	0.3295
0.156	0.3118	5.000	0.3475
0.313	0.3161	10.000	0.3494
0.625	0.3214	20.000	0.3729

(Appendix H continued)

E_o = .881 Dry Density = 88 pcf Moisture Content = 18%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.000	0.2000	2.500	0.3502
0.156	0.2555	5.000	0.3728
0.313	0.2786	10.000	0.3952
0.625	0.3011	20.000	0.4176
1.250	0.3285		

E_o = .513 Dry Density = 109 pcf Moisture Content = 15%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.1000	0.2346	0.1103
0.0147	0.1063	0.4692	0.1143
0.0293	0.1066	0.9384	0.1214
0.0587	0.1067	1.8770	0.1324
0.1173	0.1079		

E_o = .881 Dry Density = 88 pcf Moisture Content = 15%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.1000	0.2346	0.1252
0.0147	0.1008	0.4692	0.1340
0.0293	0.1013	0.9384	0.1708
0.0587	0.1046	1.8770	0.1899
0.1173	0.1124		

E_o = .491 Dry Density = 111 pcf Moisture Content = 15%

σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.1000	0.1173	0.1150
0.0197	0.1032	0.2346	0.1212
0.0293	0.1043	0.4694	0.1382
0.0587	0.1082	0.9384	0.1450

(Appendix H continued)

E_o = .887	Dry Density = 88 pcf		Moisture Content = 19%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.2000	0.1173	0.2237
0.0147	0.2009	0.2346	0.2770
0.0293	0.2016	0.4692	0.3018
0.0587	0.2070	0.9384	0.3313

E_o = .703	Dry Density = 97 pcf		Moisture Content = 18%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.0000	0.1173	0.0022
0.0147	0.0001	0.2346	0.0062
0.0293	0.0003	0.4692	0.0137
0.0587	0.0007	0.9384	0.0246

E_o = .841	Dry Density = 90 pcf		Moisture Content = 18%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.0000	0.1173	0.0030
0.0147	0.0002	0.2346	0.0114
0.0293	0.0004	0.4692	0.0284
0.0587	0.0010	0.9384	0.0494

E_o = .529	Dry Density = 108 pcf		Moisture Content = 15%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.0000	0.1173	0.0022
0.0147	0.0002	0.2346	0.0060
0.0293	0.0006	0.4692	0.0119
0.0587	0.0009	0.9384	0.0191

(Appendix H continued)

E_o = .575		Dry Density = 105 pcf	Moisture Content = 16%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.0000	0.1173	0.0032
0.0147	0.0013	0.2346	0.0064
0.0293	0.0015	0.4692	0.0120
0.0587	0.0024	0.9384	0.0194

E_o = .688		Dry Density = 98 pcf	Moisture Content = 15%
σ_v,tsf	Dial Reading,inches	σ_v,tsf	Dial Reading,inches
0.0000	0.0000	0.1173	0.0066
0.0147	0.0002	0.2346	0.0121
0.0293	0.0006	0.4692	0.0181
0.0587	0.0032	0.9384	0.0272

APPENDIX I

Summary of Sample Preparation Method Laboratory Compaction of Frozen Soils

Based on procedure in Haas, Alkire and Kaderabek (Reference 11)

This procedure was originally developed as a control test in an experiment to determine the effectiveness of additives in improving the compaction of soils at below freezing temperatures. As a control test, one of the objectives was to achieve high reproducibility of results. This objective was achieved by freezing the soil in cubes of a standardized size. After the soil to be tested had been selected and the test temperature had been selected, the following steps were used:

1) Moisture blending. Water was added to the soil in predetermined amounts to produce the desired moisture content range, typically in two percent increments. Mixing was done with a laboratory blender.

2) Formation of cubes. The moist soil was placed in plastic ice cube trays and compacted with static pressure sufficient to obtain a dry unit weight equal to that of the soil in its natural in-situ condition. The trays were selected to produce a "cube" about 0.8 inch in size, with slightly rounded edges. The trays were then placed in the cold room for freezing. The trays were supported by thick (4-inch) slabs of styrofoam, but covered on the surface by only a thin plastic sheet. As a result, the freezing of the cubes was from the surface downward. Freezing was usually complete in 24 hours.

3) Compaction. After the soil was frozen, it was removed from the ice cube trays and the frozen cubes placed in the mold of a compaction machine. The sample was compacted in layers, the method being essentially the same as conventional standard tests. The compaction was done in the same cold room in which the soil was frozen, thus preparation and compaction temperatures were the same (most of the work was done at -7°C).

4) <u>Sample Measurements.</u> Rather than trim the frozen compacted sample to the height of the mold, the sample volume was determined, using a calibrated sand cone technique adopted from the method of determining field densities. The amount of soil used was controlled so that the compacted volume did not vary appreciably from standard. Samples of the frozen soil were taken for "moisture" content as in conventional testing. Water content and dry density were calculated the same way as in a conventional lab compaction test.